STORMWATER MANAGEMENT GUIDELINES

for the Province of Alberta

Alberta
ENVIRONMENTAL PROTECTION

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Preface

Stormwater management requires Alberta Environmental Protection's approval, both under the Environmental Protection and Enhancement Act and under the Water Act. In 1987 the Standards and Approvals Division of Alberta Environmental Protection prepared the "Stormwater Management Guidelines for the Province of Alberta". These guidelines formed the basis for stormwater management in the province. Since 1987, stormwater management has taken some advances, particularly in terms of water quality. These guidelines are an update of the 1987 document and, like the previous version, are intended to help municipalities, local authorities, consulting engineers, and developers in the planning and design of stormwater management systems in Alberta. They outline the objectives of stormwater management and the available methodologies and concepts for the planning, design, and operation of stormwater drainage systems. In addition to the water quantity aspects of stormwater management, the publication also describes some of the techniques that can be applied for quality management of stormwater.

It is important that these guidelines be viewed as a tool to assist in making decisions and not as a rulebook for stormwater management solutions. The designer is solely responsible for decisions made with respect to stormwater management for any given site.

Although the guidelines provide practical and specific guidance there must be flexibility to account for site specific conditions. Stormwater management solutions are site specific and this must be recognized when applying the guidance which is provided in this document. Site-specific conditions and characteristics will govern over the guidance provided in these guidelines.

No single stormwater management technique can be universally recommended. In many instances combinations of stormwater management techniques will be required to address a range of concerns. There is limited experience with some types of techniques in Alberta, especially those involving infiltration or wetland techniques. However, their use is encouraged. In light of the limited experience with these facilities in Alberta, recurring evaluation of their hydrologic and pollutant control performance is necessary.

Ongoing maintenance, and in some cases, periodic replacement, of stormwater management facilities is extremely important to ensure effectiveness. Lack of maintenance is a primary cause of failure.

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1.0 Introduction

Over the years, the trend in Alberta has been toward increased urbanization. Both this increasing degree of urbanization and the associated higher public expectations for runoff control have been underlying forces in the trend toward the increasing use of stormwater management principles. Concerns regarding stormwater runoff and its impact on urban and rural development and on aquatic resources have also been increasing. It is generally recognized that stormwater must be addressed during the planning, design, and construction of our communities, in a different manner than in the past. To achieve development forms which meet our current needs while preserving and maintaining our natural resources for the future, it will be necessary to plan our actions in ways that recognize such things as water quality and quantity, linkages between surface and groundwater, and dependencies between physical and biological resources.

Processes and methodologies for this new type of approach are evolving in the province. Terms such as "watershed planning" and "ecosystem management", "sustainable development", "no net loss of habitat" and "enhancement" are encouraged in virtually every undertaking. In efforts to turn these guiding principles into actual applications, environmental planners, engineers, and scientists will have to use tools including source controls, conservation, land-use control, treatment, and best management practices.

To accomplish this, stormwater management practice has introduced the concept of the dual (major/minor) drainage system analysis and the use of stormwater facilities for both peak flow rate and water quality control. It has fostered the development of scientific methods that have, in part, displaced the traditional use of empirical formulae such as the Rational Method for the analysis of complex drainage systems. These changes have evolved since the mid 1960s, aided by the considerable amount of research in the United States funded by the U.S. Environmental Protection Agency (EPA). In Alberta, rapid growth in urban development during the 1970s forced developers, consultants, and the larger urban communities to reassess drainage system design practices and standards.

A high quality of stormwater management is a necessary ingredient for orderly municipal growth. Stormwater management usually requires the use of surface facilities to store, treat, or convey runoff from extreme rainfall events. These facilities must often compete for space with the other servicing components and land uses in a development. As a result, the planning of a stormwater management system must conform to the general development plan of the municipality it serves.

There has been a tendency in the past to consider stormwater runoff a liability. In the modern planning process, however, the potential for the beneficial use of stormwater should also be considered. Schemes for stormwater management facilities should consider multi-purpose applications whenever possible. Some stormwater management facilities can be aesthetic and recreational amenities for the communities in which they are located.

The development of stormwater management schemes leads to a need for the consideration of stormwater quality with respect to street cleaning and de-icing, storm sewer cleaning, solid wastes collection, and control of erosion from construction sites. In addition, facilities not specifically designed for drainage, such as parking areas, public parks, and rooftops can be used as integral components of an overall plan to control stormwater runoff. Proper consideration and implementation of this positive approach, with good policy and planning, will ensure an adequate standard of stormwater management in the future.

The purpose of these guidelines is to discuss aspects of the planning, analysis, design, construction, operation, and maintenance of stormwater management systems relevant to Alberta. These guidelines are an update of "Stormwater Management Guidelines for the Province of Alberta" (Alberta Environment, March 1987). For the most part, the updated guidelines have kept the flavour and contents of the 1987 document. However, it has been updated to include some of the advances in stormwater management, particularly those related to stormwater quality control and Best Management Practices (BMPs). In this latter class, the planning, selection, design basis, implementation, and costs of BMPs are included and discussed.

The basis for BMP selection is tied to the current, and potential, receiving-water uses. These should be established through the water shed planning and urban planning process. Once these are known, water quality objectives can be identified based on receiving-water characteristics and the BMPs selected to meet these objectives.

It should be recognized that there is no standard solution to stormwater management. Every possible solution has its advantages and disadvantages. Therefore, the selection process should be made on the basis of choosing from an arsenal of water quantity and quality improvement techniques based on water quality and quantity concerns, site conditions, capital and maintenance cost, and design experience. It should also be recognized that in many instances more than one type of technique may be required to protect a range of resources.

The use of these techniques, with other options such as housekeeping practices, land use restrictions or limitations, conservation, and enhancement programs and source controls of pollutants, will result in future development forms which provide for human needs while protecting the natural environment.

It is important that designers and others treat these guidelines as a tool to assist them and not as a rulebook for stormwater management solutions. There are many site-specific issues that affect development and stormwater management planning. Although the guidelines provide practical and specific guidance there must be flexibility to account for site-specific conditions. Stormwater management solutions are site specific and this must be recognized when applying the guidance provided in this document.

2.0 Planning for Stormwater Management

2.1 Introduction

The term "stormwater management" implies a comprehensive approach to the planning, design, implementation, and operation of stormwater drainage improvements. The purpose of the stormwater management approach is to develop effective drainage systems that balance the objectives of maximizing drainage efficiency and minimizing adverse environmental impacts.

This section outlines a basic framework for planning the development of stormwater management systems. It deals with rural and urban drainage system considerations, four levels of drainage planning (from river basin planning to implementation), and it discusses the merits of design standards for stormwater facilities.

2.2 Considerations for Rural Drainage Systems

Few drainage systems in inhabited areas remain in their natural state. The development of agriculture and of transportation networks has resulted in modifications to the natural drainage system. These modifications to land use and drainage patterns can be the source of drainage problems in rural systems. Rural and urban drainage are interrelated since both may contribute to the overall hydrology of a watershed.

The following discussion provides background for the planning of drainage systems for smaller communities that must interact with rural drainage systems.

2.2.1 Road Drainage

Most drainage improvements in rural areas relate to road drainage. Rural roads are designed to shed water into roadside ditches. In Alberta, these ditches are usually wide and shallow with flat grades, and in most cases they significantly attenuate runoff hydrographs. Most adverse impacts caused by the road system occur where they cross watercourses; ditches with steep grades approaching creeks and rivers are susceptible to erosion that may result in sedimentation problems in the stream. Road-crossing structures often restrict the flow in minor watercourses; however, in such cases it is usually the road that suffers the most damage when an extreme runoff event occurs. It is interesting that the concept of level-of-service is firmly entrenched in highway and rural road design, especially in relation to stream crossings. Invariably the capacity of such crossings in terms of flow frequency is determined by the importance of the road and the type of crossing. In contrast, this appears to be a rare consideration in the design of urban roadways. Table 2-1 shows typical design return periods for bridges and culverts for various types of roadways.

2.2.2 Agricultural Drainage

Most rural runoff in Alberta and the other prairie provinces occurs in the spring as a result of snowmelt. In most cases, significant runoff from summer rainstorms is an unusual occurrence. Since water is vital to plant growth, particularly in the form of soil moisture

retention, agricultural drainage systems are not designed to be hydraulically efficient in the rate of surface water removal. The major emphasis is on positive drainage to ensure that standing water does not remain in low areas for extended periods.

The most significant impacts of agriculture on rural drainage systems result from either the drainage of wetland areas or the conversion of woodlands to pasture or cropland. These significantly increase the amount of runoff and erosion. Where such undertakings are planned on a large scale, regional drainage planning may be warranted.

Table 2-1
Typical Design Return Periods for Bridges and Culverts

Road Classification	Return Period (years)	
	Culverts with Total Span up to 6.0 m	Bridges and Culverts with Total Span Exceeding 6.0 m
Freeway	50	100
Urban arterial	50	100
Rural arterial	25	50
Collector	25	50
Urban local	25	50
Rural local	10	25

Notes:

1. Total span for this purpose is the sum of individual spans or diameters, measured parallel to the road in the case of a bridge, and normal to the longitudinal axis in the case of a culvert.
2. The return periods listed are for guidance only and should be modified in the following cases:
   (a) If the flood hazard in the vicinity of the site is unusually severe;
   (b) If the road classification is likely to be upgraded or downgraded after construction;
   (c) If the road has an unusually low traffic volume.
3. Taken from Drainage Manual Vol. 1, RTAC (1982).

2.2.3 Urbanization in Rural Watersheds

Water quality concerns have received increasing attention over the past several decades. Initially, focus was directed at point sources of pollution, such as sewage treatment plants or industrial dischargers. Although much progress has been made in reducing or eliminating point sources of pollution, water quality degradation still occurs in some Alberta

watercourses. Accordingly, focus has shifted to non-point sources of pollution such as stormwater runoff from urban or rural sources.

A drainage system for stormwater runoff always exists, whether the land is predominantly rural or has undergone some degree of development. Virtually any modification of an existing drainage system will modify the runoff characteristics of the system.

Environment Canada has estimated that urbanization of a natural drainage basin can result in increases in stormwater runoff of 400 percent or more. Also, this stormwater carries a variety of water contaminants that may accumulate and damage aquatic environments and/or restrict water use to some degree. The impacts of urbanization vary depending upon both the scale and type of drainage improvements carried out and the sensitivity of the receiving stream environment related to each drainage system.

Where urbanization occurs in an essentially rural watershed the changes that occur in the hydrologic regime are very significant to the downstream system. Specific changes that will usually occur are:

(a) An overall increase in the annual volume of runoff,
(b) A much faster rate of runoff for any given event,
(c) Summer rainfall events that can result in significant runoff from the urbanized areas, while little or no runoff comes from the rural portion of the basin.
(d) Stream base flows (low flows) decrease

The seasonal change in the runoff pattern is particularly important. Many rural drainage channels routinely overflow their banks and cause flooding of adjacent land during spring runoff. Usually this is not a problem, provided that the excess water drains away quickly. In many cases, particularly in the upper reaches of a watershed, the primary drainage routes are not incised channels but broad overland flow routes active only during spring runoff. During the summer months these floodplains may be productive farmland where flooding would cause extensive economic damage.

The imposition of rigid flow regulation policies for rural drainage based on pre-development/post-development concepts should be avoided. It is imperative that the required capacities of discharge channels be determined on the basis of hydraulic analysis. This should include analysis of both channel capacities and the restrictions due to stream crossings. It should not be assumed that a flow rate that occurs without problems during the spring would not cause problems during the summer. In cases where high-capacity channels exist, flow regulation may be unnecessary for summer storms. In addition, such regulation may retard spring runoff to the extent that it coincides with upstream peak flows resulting in higher discharge rates than would otherwise occur.

2.3 Drainage Considerations for Urban Areas

2.3.1 The Impact of Development

From a drainage perspective the most dominant characteristic of the urban landscape is the high degree of impervious ground cover. Urban areas are also characterized by systematic surface grading intended to direct the flow of water. A secondary effect of grading is to ensure a rapid rate of runoff. These factors result in more significant changes to the hydrologic regime in comparison with changes due to drainage works in rural areas.

Runoff events occur on a routine basis in urban areas. Summer rainstorms that would produce little or no runoff in rural areas produce significant runoff in urban areas. The overall effect of large amounts of impervious land surface in urban areas is to dramatically increase the runoff volume and the rate of runoff from each rainfall event. In contrast to rural areas, the runoff produced by snowmelt events in urban areas rarely exceeds the drainage system capacity.

2.3.2 Convenience Drainage for Urban Areas

Traditionally, drainage planning for urban areas has focused on the inconvenience caused to human activity by runoff. As a result, urban drainage systems are characterized by positive grading of land away from buildings to roadways which in turn are graded to ensure rapid flow of surface water to a point where it can be discharged to an underground sewer system. The sewer system conveys the water to a point where it can be discharged to a watercourse or water body. The hydraulic efficiency exhibited by such systems under normal circumstances is primarily due to the storm sewer system. In addition to hydraulic efficiency, sewers use less additional land and require a minimal amount of maintenance in comparison to other municipal services.

Storm sewer systems are generally considered to be expensive, and the fact that the larger, more costly trunk sewer components have to be installed prior to surface improvements results in a high initial or front-end cost. Economic considerations effectively limit the hydraulic capacity that can be provided by storm sewer systems. It is common practice to design these systems to discharge the flow generated by rainstorms with a 1-in-5-year return period (rainstorms that have a 20% chance of occurring every year).

2.3.3 Flood Protection in Urban Areas

While storm sewer systems work effectively under normal circumstances, occasional runoff events will occur where the system has insufficient capacity to discharge in the normal manner. The impacts of the runoff rate exceeding the capacity of the system can vary from temporary accumulation of standing water around a street inlet to large volumes of water flowing overland, causing flooding of buildings and other facilities. Situations occur in some systems where water may flow out of manholes and inlets in the downstream parts of the system onto streets and adjacent property. Until recent times it appeared that little or no

consideration was given to what would happen when the system capacity was exceeded. However, in recent years developers have been required to design overland flow routes to prepare for these excess flows.

With the passage of time, it has become apparent that the degree of flood protection provided by urban drainage systems designed primarily for convenience is not adequate. Provision must also be made to control the excess runoff resulting from infrequent events.

2.3.4 Storage for Flood Protection

Stormwater storage has proved to be a flexible approach to mitigating many urban drainage problems. Storage facilities include underground vessels (superpipes), temporary flooding of open-space areas (parks, parking lots, etc.), and storage ponds. The most common form of storage is the stormwater detention or retention pond. These facilities have become synonymous with stormwater management.

The introduction of a storage element into a drainage system serves to attenuate the hydrograph flowing through it, in effect reducing the peak flow rate. The attenuation capability, more commonly referred to as routing effect, is inherent in all hydraulic systems that transport intermittent or varying flows. Routing effects are smaller for efficient hydraulic sections such as pipes than for less efficient hydraulic sections such as natural channels.

Buried storage has limited application because of the relatively high costs involved. Impoundment of stormwater in large surface depressions is cost-effective in many cases, especially in newly developing areas where land can be made available. The location of an impoundment in conjunction with a natural drainage route greatly facilitates the control of excess runoff during major events.

Most stormwater impoundments are classified as detention storage facilities. These facilities temporarily store water in excess of the downstream conveyance capacity. Detention storage facilities can be either normally dry or retain a permanent wet water body. As hydraulic facilities, both function in an identical manner. Retention storage facilities store water for extended time periods and may rely entirely on evaporation and infiltration for ultimate disposal of runoff. Retention storage facilities are not commonly used in urban environments (Note: others define detention and retention differently; using the above definitions, the majority of urban storage facilities are termed either dry ponds or wet ponds).

2.4 Planning Levels for Stormwater Management

2.4.1 Introduction

The benefits of drainage improvements tend to be readily apparent and to be localized to the site of the drainage improvements. Adverse environmental impacts generally occur downstream of the improvement site and are not always readily apparent. These impacts are of concern as they tend to be accumulative over both time and space. It would be impractical to evaluate the environmental impacts of all localized drainage improvements within a watershed on an individual basis. Local improvements should be made in the context of an

overall plan for the entire watershed. As a result, a hierarchical approach to the planning and implementation of stormwater management is required.

2.4.1.1 Hierarchical Planning Approach

Stormwater management planning must be carried out on a scale that allows assessment of impacts throughout the natural drainage system. In Alberta, the major river basins are used as macro planning units. These units are too large to be practical units for drainage planning; however, they may identify some constraints for smaller planning units.

At the other end of the spectrum, most drainage improvements are designed and implemented to service residential or industrial subdivisions of a few hectares. Land development and land-use planning units are primarily influenced by legal boundaries, land ownership, and political jurisdictions. These units rarely coincide with natural drainage boundaries, and are usually too small for stormwater management planning.

It is evident that intermediate levels for stormwater management planning are required. The four levels of planning and design for stormwater management systems that meet the requirements include major river basin plans, watershed drainage plans, master drainage plans, and site implementation plans.

The geographic relationship of these levels of planning are illustrated in Figure 2-1. The functional relationship of these levels of planning is illustrated in Figure 2-2.

2.4.2 River Basin Plan

River basin planning typically considers the major river basins in the province. River basin planning is basically concerned with the supply and demand for water as a resource. The impact of urban land drainage on the hydrology of Alberta's major rivers is negligible for most practical purposes due the small portion of urbanization in the river basins. However, Urban and industrial developments do have a significant impact on the water quality of these rivers.

It is probable that urban storm drainage systems contribute a significant portion of the pollutant loads found in Alberta's rivers. It is possible that future restrictions on the quality of stormwater discharges may be imposed as a result of investigations at this level of planning. It may also be logical to assess the need for regional drainage plans at this planning level. In these cases, boundaries for such regional studies would be defined based on anticipated development.

This level of planning is essentially a provincial responsibility.

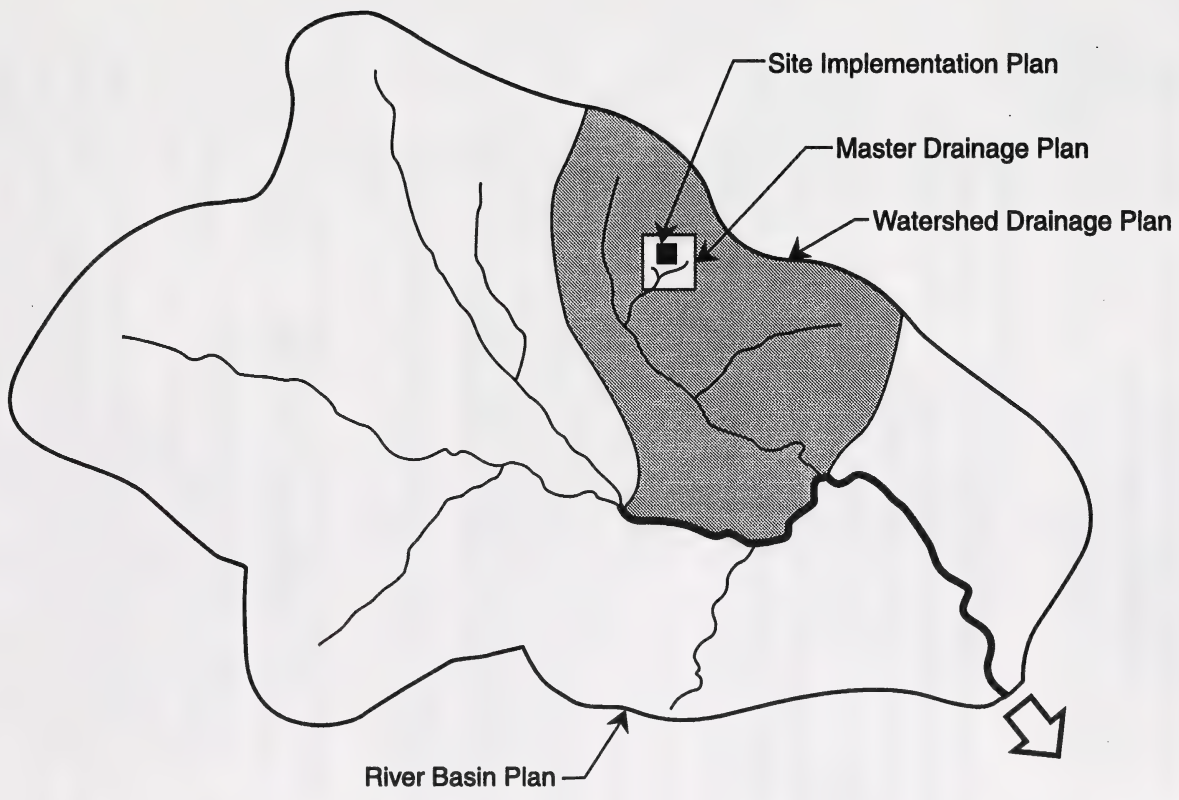


Figure 2.1
Stormwater Management Planning

River Basin Plan

- Considers Entire River Basin
- Assessment of Water Demand
- Assessment of Water Quality
- Assessment of Water Supply

Water Drainage Plan

- Considers Functional Watershed Area
- Assessment of Environmental Concerns
- Assessment of Future Drainage Requirements
- Assessment of Flood Protection Levels
- Establishment of Environmental Constraints
- Assessment of Future Drainage Requirements
- Establish Policies and Criteria for Drainage Improvements

Master Drainage Plan

- Considers Functional Stormwater Management Area
- Identifies Local Environmental Resources
- Identifies Control Levels for Stormwater BMPs
- Identifies BMPs

Site Implementation Plan

- Considers Specific Development Area
- Detailed Design of BMPs
- Approvals Obtained for BMPs
- Mitigative Measures Established for Environmental Protection
- Construction of Stormwater Control Facilities

Figure 2.2
Functional Relationship of the Four
Levels of Stormwater Management

2.4.3 Watershed Drainage Plan

2.4.3.1 General

Watershed planning has become more prevalent in recent years as a planning approach that compatibly integrates natural systems and land-use change. Watershed planning considers similar environmental issues as traditional subdivision or site planning, but at a scale where ecosystem functions and linkages can be identified and the cumulative impacts of development/resource management strategies evaluated. The watershed/subwatershed planning process provides key direction to stormwater managers in the following areas:

- Identification of constraint areas
- An analysis of the cumulative impacts associated with urban development
- Recommendations for land-use restrictions and development criteria

Watershed-wide management options usually include structural (for example, urban stormwater BMPs, servicing options, remedial projects) and nonstructural (for example, policies, programmes, bylaws) recommendations. A watershed plan will normally specify performance criteria that a stormwater management plan (SWMP) should be designed to satisfy. Such performance criteria could include:

- The requirement for lot level controls and site planning techniques to promote infiltration and maintain the water balance

- Allowable types of end-of-pipe stormwater management facilities

- Approximate locations for end-of-pipe stormwater management facilities

- Required levels of control (storage or recharge volumes/retention times) for flood control, water quality maintenance, and erosion protection

- Special SWMP design requirements (that is, thermal mitigation measures, enhanced nutrient control, spill capture/control enhancements)

- Requirements for special-purpose SWMPs: (for example, oil/grit separators for specified land uses, disinfection processes)

Basic to the development of an effective stormwater management plan is the definition of areas within which the cause and effect relationships between individual drainage elements may be significant. A watershed drainage plan should consider drainage requirements and potential impacts on a regional basis and develop a SWMP at a conceptual level. A watershed drainage plan, therefore, may consider the drainage requirements of a single drainage basin or a group of sub-basins which contribute to a point in the natural drainage system.

Planning and implementation of storm drainage is traditionally a municipal government responsibility. Factors inhibiting development of watershed drainage plans have been the initial study costs, coupled with few short-term benefits. In many cases drainage basins cover more than one municipal administration.

Drainage problems created by development are not always visible in the early stages of development. The lack of comprehensive drainage planning in such cases invariably leads to long-term flooding and/or pollution problems. This has been demonstrated by many river systems in the United States. It is noteworthy that funding by the U.S. government for the relief of such problems is increasingly being made subject to the development of comprehensive stormwater management plans and administrative structures to implement them.

This level of planning likely requires joint participation of both provincial and municipal governments.

2.4.3.2 Development of A Watershed Plan

The development of an integrated stormwater management plan requires the definition of an area within which the interactions between individual elements of the drainage system may be significant. Where large-scale land changes will occur within a drainage basin, watershed drainage plans should be developed through comprehensive studies of the existing systems and future drainage requirements. Such studies should comprise:

- A detailed analysis of the existing drainage system and identification of environmental concerns, and

- An analysis of future drainage requirements and options available for meeting those requirements.

There are no standards to determine the scope of a watershed drainage plan. The downstream area of influence of land-use changes may extend beyond the local drainage basin. Logically, the area covered by the plan should extend to where potential impacts are negligible, or where higher-level studies or statutory regulations have established limits for the impacts of surface runoff on a receiving stream or water body.

The basic objectives of a watershed drainage plan should be:

- To provide an acceptable level of flood protection for existing land uses throughout the basin,

- To establish constraints within the system to prevent environmental damage, and

- To develop policies and design criteria for use in the development of comprehensive plans for drainage improvement that may be required in the future.

Watershed drainage plans are usually developed at a conceptual level, at least where the drainage basin is largely undeveloped. The basic elements of a watershed drainage plan should include:

(a) A statement of objectives,
(b) Delineation of the watershed(s),
(c) Delineation of internal sub-basins,
(d) Identification of the constraints governing the development of the plan,
(e) Location and capacities of the principal drainage routes,
(f) Identification of major urbanization impacts,
(g) Formulation of an optimum conceptual drainage plan for the basin,
(h) Guidelines for implementing the plan, and
(i) Criteria for designing system components which will meet the objectives.

Basic data required to develop a watershed drainage plan include:

(a) A long-range land-use plan,
(b) A biological inventory of the drainage basin,
(c) A set of constraints for potential environmental impacts, and
(d) Methodologies for carrying out hydrologic, hydraulic, and (possibly) water quality analyses.

The format of a watershed drainage plan and the level of effort required to develop it will vary from system to system. In a drainage basin where significant land-use changes are not expected, the plan may be a simple document that delineates the plan boundary and sets out an approval procedure for local drainage improvements based on conservative assumptions. In addition, the plan should set out requirements or guidelines for detailed studies if a need arises, perhaps by reference to a standard document.

2.4.3.3 Watershed Drainage Plan Activities

Initial Hydrologic Activities - These include defining basin boundaries, physical characteristics, and estimating the runoff regime for existing conditions. Estimation of pre-development runoff is a fundamental component of the drainage plan. To control post-development runoff, the pre-development flow rate peaks and durations must be known to allow adequate sizing of storage facilities. Streamflow records are the only reliable data source for estimating runoff. Runoff models must be calibrated to the streamflow records to be realistic and accurate. Unfortunately, comprehensive streamflow records are rarely available. Various techniques are available for synthesizing flow data and are discussed in subsequent sections of this report.

The hydraulic capacity of the existing drainage facilities should also be estimated, where applicable. Typically, the system capacity is limited by one or more restrictive elements such as road crossings or channel constrictions imposed by existing land uses. The benefits of eliminating local constrictions should be considered.

Flood Levels - Once the watershed hydrology and hydraulic constraints are established, floodplain delineations to determine flood lines for various flow frequencies under existing conditions can be determined. It should be recognized, however, that delineating a floodplain based on existing conditions will be inadequate if future urbanization increases flood flows.

It is common practice, and accepted policy in Alberta, to define flood protection requirements in terms of a flood with a 1-in-100-year return period. Therefore, it would appear to be redundant to estimate flood lines for lesser return periods, although considerable caution should be exercised in this regard. The definition of a design flood line can have a major impact on the overall economics of a project. If the initial flood level estimate is overly conservative, a viable project may be abandoned. Conversely, if the initial estimate is too low, unexpected costs may be incurred in the future.

The degree of reliability that can be placed on an estimate of the amount of runoff generated by a rare event will vary depending upon the reliability of the base data used to make the estimate. The estimation of higher frequency (common) events is inherently more reliable than for low frequency (rare) events, principally because they can be related to experience and observation. A series of flood lines provides a basis for evaluating both the reliability of an extreme flood projection and the sensitivity of the development costs.

Constraint Identification - At this point other sites sensitive to potential adverse impacts should be identified, including sites with erosion or sedimentation potential, and biological communities.

In the quest to preserve the natural environment to the greatest extent possible, it is becoming the norm for authorities to assemble biological inventories of lands within their jurisdictions. Where up-to-date inventories do not exist they may be required as was the case in recent watershed drainage plans carried out by the City of Edmonton. Such an environmental inventory may include the identification of archaeological sites, soils data, geological data, geotechnical data specifically relating to streamcourse stability, quantification of both upland and wetland habitat, and an inventory of existing biota in the system.

Potential impacts are not solely attributable to stormwater management practice. If a biological community will not survive the proposed changes in land use, then potential adverse impacts due to drainage modifications are irrelevant. Direct stormwater management impacts are usually limited to the immediate environs of natural watercourses and water bodies. Specifically, these would include changes in the flow regimes (especially seasonal changes), streamflow velocities, and water quality.

Future Flow Conditions - Methods of estimating the magnitude and frequency of future flows occurring in the basin must take into account the effect of increased imperviousness of the land surface, and the effects of improved surface-grading and improved channelization. Peak flow rates alone are not indicative of the total change in the hydrologic regime which would occur in the drainage basin. Changes in runoff volume are equally and often more important. As well, natural environments can be more sensitive to changes in average conditions than to the short-term effect of occasional high-flow conditions.

Once the future hydrologic regime has been estimated, problem areas can be identified and mitigative measures can be devised and evaluated. Mitigative measures fall into two categories: channelization and flow regulation.

Channelization includes deepening, widening, and straightening of natural channels or the construction of diversion channels to accommodate upstream inflows. In some cases, floodproofing of specific sites by diking and pumping may increase the capacity of existing channels. Channelization commonly destroys streams and riparian habitats and degrades water quality. As an alternative to channelization, channel enhancement can be used for redesigning the stream channel. This can enhance flood conveyance without destroying habitat or aesthetic values.

Flow regulation techniques use temporary impoundment of runoff to reduce flows to rates that can be accommodated in a downstream system.

Economic Analysis - Economic analysis of stormwater management alternatives is a major consideration in developing a watershed drainage plan. While benefit-cost analysis is the most comprehensive approach to identify the best solution, it is often difficult to carry out. This is especially the case if there are significant non economic costs or benefits associated with the stormwater management plan. The aesthetic values of retaining natural vegetation or of creating an artificial lake do not lend themselves to cost/benefit analyses. The benefits of incremental levels of flood protection are also difficult to assess. While recognized methodologies are available to estimate flood damages, they involve a major effort which is generally out of proportion to the overall planning effort.

Probably the most appropriate approach to the economic evaluation of management plans is to determine the cost-effectiveness of alternatives, based on the tangible aspects. This approach requires that intangible environmental aspects be initially evaluated by some empirical process, and those aspects deemed to be important are then included as objectives of the plan. The mitigative measures for the important environmental issues then become a requisite part of the watershed drainage plan.

The cost-effectiveness of an alternative should be evaluated on the basis of total costs, monitoring and environmental, over a long term and should include both capital costs and operating and maintenance costs. The major advantage of this lifecycle approach is that effects of staging the implementation of components of the plan in step with the staging of development can be properly evaluated.

2.4.4 Master Drainage Plan

2.4.4.1 General

The purpose of a master drainage plan is to ensure that the optimal drainage system is developed to meet present and future requirements. The drainage area included would either be determined by existing drainage boundaries or boundaries imposed by a watershed drainage plan. These drainage areas would not be based on jurisdictional boundaries. The master drainage plan is developed through the evaluation of alternatives that provide an

acceptable level of service while meeting the objectives of the watershed drainage plan and satisfying constraints imposed by topography, existing and proposed land uses, land ownership, and other local considerations.

The minimum scope of the master drainage plan is to identify and locate major drainage facilities, including trunk sewer routes, open channel routes, storage facilities (and their associated floodplain levels and areas), and land requirements for drainage purposes. If sufficient land-use planning information is available, preliminary designs of the major facilities may be developed in the plan.

Many jurisdictions encourage regional master drainage planning for developing areas. The master drainage plan level for stormwater management will enable an integrated approach to stormwater management planning. The objectives of a master drainage plan will typically be to:

- Identify specific local resource of regional significance to be protected.
- Specify the size, type, location, and performance characteristics of regional stormwater BMP facilities.
- Identify the requirements for and performance characteristics of local BMP plans based on uses to be maintained. Specific design targets (that is, infiltration, peak runoff, retention time, temperature) should be set.
- Specific objectives for identified SWMPs (that is, bacteria control, oil/grit separators).
- Identify requirements for regional and/or local systems and online or offline systems.

Master drainage plans should develop alternatives and identify optimal drainage solutions that conform with the objectives of the watershed drainage plan and the realities of the proposed land-use plan. The scope of the master drainage plan generally covers a portion of the area served by the watershed drainage plan such as one or more sub-basins. Ideally, it complements a "neighbourhood structure plan".

Although this level of planning is basically a municipal responsibility, the development of such a plan may be undertaken by private developers with large land holdings within the plan boundaries.

2.4.4.2 Master Drainage Plan Activities

The first step is to define the existing local drainage system. Where a natural drainage system is well defined it will invariably become the major drainage system. In such cases it will rarely be economical to introduce radical topographic changes that significantly modify the natural drainage system.

In many parts of Alberta the natural drainage system can be classified as poor. In such cases of poorly drained land, the natural drainage is internal toward potholes, sloughs, or marshes. Surface discharges from these collection points are rare or nonexistent. In many cases, the sloughs, marshes, etc. may be effectively integrated into the drainage system.

The second step is to assess the need for discharge controls to meet downstream hydrologic constraints identified in the watershed drainage plan. Increased in-system storage may be required to control discharge rates. This can be accomplished by using existing creeks or enhanced channels and floodplains, using artificial storage lakes, or by retaining existing sloughs or marshes. Storage facilities may also be beneficial for economic reasons, such as reducing the size of long outfall sewers. This can achieve considerable cost savings.

Where in-system storage is required it must be located to intercept overland flows from major events as well as flows from the more common events. Once storage facilities and the major system routes have been located, major trunk sewer routes can be determined. This can be followed by the location and design of the minor system piping.

2.4.4.3 Land for Stormwater Storage

Provision of land required for stormwater storage lakes may sometimes cause problems in allocating development costs or where development of fragmented land holdings is concerned.

One particular problem in Alberta is that the Planning Act does not provide a zoning category for stormwater management lakes that is equitable to both the municipality and the private developer. One allowable classification is as a Public Utility Lot (PUL). This option will penalize a developer with a storage facility located on his/her land, as PULs are dedicated from net area and are subject to all municipal service levies. The only other allowable classification is that of Municipal Reserve (MR). The Planning Act limits MR to 10 percent of the gross developable area. Storage lakes would generally absorb 20 to 30 percent of this area, which in most cases would be quite inequitable from a municipal perspective.

The status of existing wet detention pond sites in Alberta has generally been resolved either through a negotiated agreement between the developer and the municipality, or by having ponds constructed on municipal land. Typically, the normal water surface area has been treated as if it were MR, although it cannot be zoned as such. This in effect has reduced the gross developable area from which other dedications and levies are determined.

Problems may arise when a drainage plan identifies a site required for a storage facility. If the land holding is small enough that the facility restricts the developability of the parcel, it devalues the individual property; conversely, a landowner may be in a position to hold a municipality to ransom. Thus, the implications of land requirements should be a specific consideration during the development of a master drainage plan.

2.4.5 Site Implementation Plans

At this level, the detailed design and construction of drainage facilities as defined in the master drainage plan are carried out. During the implementation phase of a stormwater management plan, extensive environmental damage can result from land-development activities (for example, surface and stream erosion and subsequent sedimentation in downstream waterbodies). Mitigative measures should be implemented during these operations.

2.4.5.1 Design Standards for Site Implementation Plans

Many communities have developed design standards that establish the requirements for their infrastructure. These deal with the design and construction of roads, curbs and gutters, sidewalks, water pipes, sanitary sewers, storm sewers, service connections, power, telephone, and cable television. Although not all communities have standards that deal with all of these utility systems, most have standards that deal with sanitary and storm sewers. These can range from a concise statement of the physical and design requirements for the sewer systems to the identification of a comprehensive systems planning approach. The latter category covers all activities (from developing watershed drainage plans through to construction), to which a developer and the municipality must conform to implement a subdivision.

Although design standards are limited to scope, they are valuable in that they present the philosophy of the municipality with respect to stormwater management. They provide the minimum levels of service and analysis expected, and where more than one option for design is possible, they clarify local preference. Design standards also serve to identify points of local importance that might be overlooked (for example, the groundwater table, soils characteristics and location of the 100-year floodplain for the local streamcourse).

Design standards have been criticized because they tend to discourage innovation and are unable to address site-specific problems (for example, soil foundation conditions and sulphate content). In considering this limitation, it has been suggested that the following clause should preface all design standards:

"These standards shall not be considered as a rigid requirement where variation will achieve a better technical and/or economical solution. Indeed it is encouraged that consulting engineers continuously seek new and better solutions."

This flexibility is of value for larger communities that either retain consultants or have staff who can review in detail the technical content of development proposals and design submissions. However, where the commitment to provide this review is not possible, or where the magnitude of the development is small, the use of a reasonable set of design standards should be required. With proper application, design based on such standards will provide an adequate level of service.

For many items, acceptable standards can be achieved by requiring one of several options. The items and standards given are intended to reflect what is felt to be the most appropriate or most common practice in Alberta. The local authority is left with the authority as to the selection of the details of the design standards.

2.5 Stormwater Drainage Planning Outside of a Watershed Context

2.5.1 General

Many environmental impacts associated with development cannot be addressed at the plan-of-subdivision or site-plan level. As discussed above, the watershed/subwatershed planning process is a valuable tool in guiding land-use planning and performance criteria in a larger, ecosystem-based context. However, it is recognized that, in many cases, development will proceed in the absence of watershed/subwatershed or master drainage planning. In such cases, the stormwater management planning and selection process can be collapsed into a single-stage process shown in Figure 2-3. This single-stage process, although necessary in many circumstances, is not recommended for the following reasons:

- Watershed/subwatershed ecosystem and water management issues and priorities are not identified.
- Cumulative impacts of development on flooding, water quality, erosion, and baseflow cannot be assessed.
- The identification of natural area linkages and wildlife corridors is best accomplished at the watershed/subwatershed scale.
- Regional facilities cannot be evaluated.

This section describes an approach to stormwater management planning and design where guidance is not available from a watershed/subwatershed or master drainage plan. The approach has three components:

- Design objectives and information requirements
- Assessment of receiving-water concerns
- Selection of SWMP criteria
- SWMP selection

2.5.2 Design Objectives and Information Requirements

2.5.2.1 Design Objectives

Stormwater site planning is a fundamental determinant in how a given development will impact the hydrologic cycle. All new development, whether addressed through a watershed/subwatershed plan or not, needs to be properly planned to ensure that deleterious environmental impacts are minimized. Effective subdivision/site planning is a necessary prerequisite for effective stormwater management.

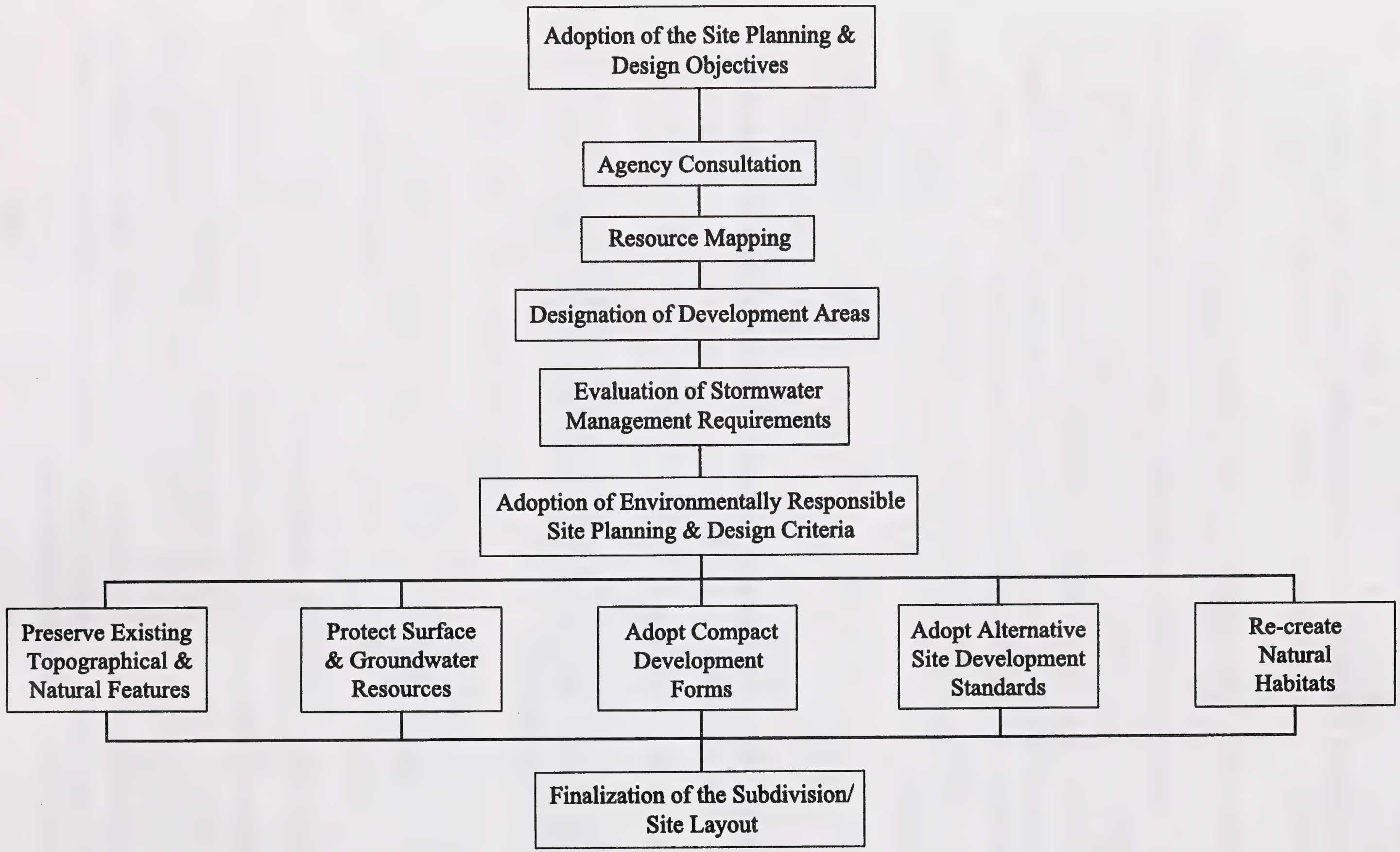


Figure 2.3
Subdivision/Site Planning Methodology

Five design objectives guide the relationship between subdivision/site planning and stormwater management:

- Reproduce pre-development hydrological conditions.

- Confine development and construction activities to the least critical areas.

- Maintain the overall desired density of development by allocating higher densities to areas suitable for development (if required).

- Minimize changes to existing topography.

- Preserve and utilize the natural drainage system.

2.5.2.2 Agency Consultation

The provincial and federal regulatory agencies (Alberta Environmental Protection and Environment Canada) should be contacted to assist in the identification of:

- Existing natural resource mapping/data
- Local natural resource issues or concerns

2.5.2.3 Resource Mapping

Important natural resources need to be identified and mapped before final layouts and design can proceed. The following resources should be mapped if applicable:

- Watercourses, waterbodies
- Stream and valley corridors
- Flood and/or fill lines
- Wetlands
- Areas of natural or scientific interest
- Environmentally sensitive areas
- Significant vegetation/woodlots
- Wildlife habitat/corridors
- Significant groundwater recharge areas
- Steep sloped areas
- Erosional areas

2.5.2.3 Evaluate Stormwater Management Requirements

Once significant natural resource areas have been identified, remaining areas can be classified as "developable areas". In evaluating stormwater management requirements, receiving water concerns need to be identified and water management criteria developed

before alternative management strategies and land area requirements can be evaluated in finalizing subdivision/site plans.

2.5.3 Assessment of Receiving-Water Concerns

Four water quality criteria, or receiving-water concerns, generally guide the application of SWMPs:

- Water quantity (flooding)
- Water quality (pollutant loading, aquatic habitat, recreation, aesthetics)
- Erosion potential (in-stream erosion)
- Baseflow (groundwater recharge, in-stream low flow maintenance)

The above criteria/concerns may be affected by land-use change. Their maintenance helps ensure receiving waters can support an appropriate diversity of life and do not undergo damaging geomorphological change.

2.5.4 Selection of SWMP Design Criteria

In the absence of watershed or master drainage planning, specific performance criteria need to be established to guide the application of local SWMPs. These design criteria are developed for each category of concern recognized in the receiving water.

2.5.4.1 Water Quantity

Ideally watershed/subwatershed or master drainage plans should evaluate requirements for post-development water quantity controls based on the potential cumulative impacts of development and potential flood hazards. Where higher-level water management plans do not exist, requirements for water quantity control will be based on potential downstream flooding hazards.

Water quantity control is generally most effective in the headwaters of a watershed/subwatershed. Extended detention does not substantially decrease storm runoff volumes, but reduces peak flow rates or elongates hydrographs. Where extended detention is used in downstream reaches, water released from upstream reaches may "add" to that released from downstream storage, resulting in peak flow durations greater than those experienced pre-development. Also, in arid areas it has been observed that extended open water evaporates sufficiently to affect annual volumes and therefore the design of the stormwater management facilities.

Although site-specific characteristics should govern appropriate water quantity controls, the following general recommendations are made:

- If there is a potential flood hazard directly downstream from a proposed site, water quantity control must be implemented.

- If the development is located in the headwaters area, the post-development peak flow rates should be controlled to pre-development levels.

- If the development is located in the lower reaches of the watershed/subwatershed, quantity control may not be required. This is because runoff from the lower reaches of the watershed may have discharged before the main flood arrives.

Typically water quantity targets aim to control post-development peak flows for the 2- 5- 25- and 100-year storms to pre-development levels.

2.5.4.2 Water Quality

A required storage volume can be established based on the sensitivity of the receiving water aquatic habitat and the level of development or imperviousness proposed. For example, in Ontario, aquatic sensitivities or "levels of protection" required are based on the Ontario Ministry of Natural Resources'(MNR) 1994 "Fish Habitat Protection Guidelines For Developing Areas". Other provincial and state governments have similar guidelines. Table 2-2, which is taken from the Ontario Stormwater Management Practices, Planning and Design Manual developed in 1994 by the Ontario Ministry of Environment and Energy (MOEE), presents examples of water quality control criteria for meeting these levels based on receiving water sensitivities and levels of development. So far in Alberta, stream classification has not been carried out. However, the Ontario examples discussed below can be used as a guide for determining storage requirements. Storage requirements may need to be increased to accommodate snowmelt for the conditions in Alberta.

<u>Level 1 Protection</u>

Type 1 habitats support the overall fisheries productive capacity and include:

- Spawning areas for species with stringent spawning requirements
- Essential rearing areas
- Highly productive feeding areas
- Refuges
- Constricted migration routes
- Habitats supporting endangered, threatened, or vulnerable species
- Groundwater recharge areas supporting cold water streams

Level 2 Protection

Type 2 habitat is generally abundant and is not the limiting factor for a species' productive capacity. Examples include:

- Feeding areas
- Areas of unspecialized habitat
- Pool-riffle-run complexes

Level 3 Protection

Type 3 habitat has a low capacity for fish production and does not have a reasonable potential for enhancement. Examples include:

- Municipal drains
- Highly altered, hardened, or polluted watercourses
- Artificial drainage swales

Level 4 Protection

Level 4 protection is the minimum level of water quality control acceptable and is intended only for retrofit and redevelopment situations.

Recreational Concerns

Recreational activities that involve water contact, such as swimming, may require additional upstream water quality controls such as ultraviolet (UV) disinfection. The requirement for additional controls will depend on the impact of the stormwater discharge on the recreation area. The impact is related to the distance between the development and the recreation area and the size of the contributing drainage area upstream of the recreation area compared to the size of development. Water quality monitoring programs may be desirable to determine the level of impact.

Temperature Concerns

Urbanization causes an increase in the temperature of runoff waters. The storage and discharge of runoff through ponds can further increase water temperatures downstream. In areas where there is a temperature concern in receiving waters and no higher-level plan is in place, the following mitigative measures may be implemented:

- Stormwater lot level controls are maximized.
- Outlet cooling is provided.
- A vegetative strategy is implemented to provide maximum shading.
- Facility configuration designed such that large, open areas of water are minimized.
- Alternative site planning techniques are investigated.

Table 2-2
Water Quality Storage Requirements Based On Receiving Waters [1,2]

Protection Level	SWMP Type	Storage Volume For Impervious Level (m^3/ha)			
		35%	55%	70%	85%
Level 1	Infiltration	25	30	35	40
	Wetlands	80	105	120	140
	Wet pond	140	190	225	250
	Dry pond (batch)	140	190	210	235
Level 2	Infiltration	20	20	25	30
	Wetlands	60	70	80	90
	Wet pond	90	110	130	150
	Dry pond (batch)	60	80	95	110
Level 3	Infiltration	20	20	20	20
	Wetlands	60	60	60	60
	Wet pond	60	75	85	95
	Dry pond (batch)	40	50	55	60
	Dry pond	90	150	200	240
Level 4	Infiltration	15	15	15	15
	Wetlands	60	60	60	60
	Wet pond	60	60	60	65
	Dry pond (batch)	25	30	35	40
	Dry pond	35	50	60	70

Notes:

1. From MOEE Stormwater Management Practices, Planning and Design Manual, 1994

2. For wetlands and wet ponds all of the storage, except for 40 m^3/ha, represents the permanent pool volume. The 40 m^3/ha represents extended detention storage. All values are based on specific design parameters and 24 hour detention.

2.5.4.3 Erosion Potential

In the watershed planning process it is normal practice to estimate the erosive potential of a variety of storms under different development conditions. The preferred approach for addressing erosion concerns is at the watershed/subwatershed planning level through the use of continuous simulation to determine the pre- and post-development exceedance of a watercourse's "erosive index" (based either on tractive force or velocity-duration data). At the development level, a similar tractive force/permissible velocity analysis can be applied using a design storm of a specific depth over the catchment. In Ontario, for example, the design storm is the 4-hour Chicago Distribution 25-mm storm. The derived extended detention volume suitable for erosion control should be compared to the water quality control criteria and the greater of the two should be used for design.

Erosion is, however, a complex natural process that is difficult to predict. The use of a design storm of a specific depth in place of a continuous simulation does not take into account change to the hydrologic cycle (that is, flow durations) induced by development or pre-development conditions of the receiving water stream (that is, bank stratigraphy, channel slope, or channel cross-section). A level of confidence in the use of a storm of a specific depth as design criteria for erosion control can be obtained by determining the erosive index of the stream under pre-development and post-development conditions for the design storm event. Comparison of the two indexes will provide a relative assessment of the change in erosive potential.

2.5.4.4 Baseflow Maintenance

Watershed/subwatershed plans are also the preferred level of study for integrating recharge concerns with stormwater management. In the absence of this level of planning, as a minimum, no runoff from a 5-mm storm should occur from any development (excluding roads).

SWMP options may include:

- An emphasis on site planning opportunities
- Maximum use of on-lot and conveyance controls
- Use of infiltration techniques where appropriate

2.5.5 SWMP Selection

The goal of stormwater management is to preserve the natural hydrologic cycle. Watershed/subwatershed plans will recommend appropriate SWMPs, including the volume of control required, to ensure the cumulative impacts of development (flooding, water quality degradation, erosion problems, loss of baseflow) are minimized. In the absence of watershed/subwatershed planning, SWMP selection should still aim to maintain the natural hydrologic cycle, thus minimizing potential impacts to downstream water and land resources. In trying to maintain the natural hydrologic cycle, the following approach should be taken in selecting appropriate SWMPs:

- Screen stormwater lot level controls for implementation.
- Assess stormwater conveyance controls for implementation.
- Implement end-of-pipe stormwater management facilities to address remaining concerns.

3.0 Drainage Systems for Urban Areas

This section describes the various elements that make up a typical urban drainage system. The basic concept of dual drainage, level of service, design capacities, and stormwater runoff control alternatives are described. The application and design of Best Management Practices (BMPs) to improve the operating functions of these drainage elements are described in Section 6. A proper urban drainage system design integrated with appropriate BMPs, either as stand alone facilities or in combination, will provide a solution to the stormwater management concerns at a particular site.

3.1 The Dual Drainage Concept

Traditional stormwater drainage systems have consisted primarily of underground pipes and related works designed to transport flows for relatively minor rainstorms. "Minor" in this context means the most severe storm that would be expected to occur, on average, once in a period of (for example) 5 years. Although return periods varying from 2 to 10 years are used as design standards in different municipalities, the most common design criteria for the underground pipe system has been the 5-year-return-period storm. Historically, there was little or no consideration given to controlling the runoff from larger storms. During major events, meaning larger than the 5-year storm, numerous flooding problems typically occurred.

The solution to the problems that occurred during these infrequent but large events has been to make allowances for major events in the planning and design of new land developments. The division of storms into minor and major events and the realization that separate systems (the minor and major systems) are required for each became known as the "dual drainage" concept (Figure 3-1). The minor system provides a basic level of service by conveying flows from the more common events. The major system conveys runoff from the extreme events in excess of the minor system capacity (providing flood protection usually up to a 100-year-return-period storm). There is always a major system, whether or not one is planned. Failure to plan for a major system often results in unnecessary flood damage. It is now common practice to provide for a major system during the design of any land development. This does not necessarily imply an expensive stormwater management study; it is often only necessary to examine development grading plans to ensure that there is an overland flow path with reasonable capacity.

Some municipalities require that flow depths and velocities be calculated in the major system. This requires a separate calculation for the major and minor systems. OTTSWMM, and its update DDSWMM, is an example of a computer model designed to facilitate this type of calculation.

Good planning and design integrates the design of a site and the design of the stormwater management facilities into one process. Similarly, the integration of BMPs into the planning and design process of the drainage system is essential if an effective stormwater management plan is to happen. This section presents an introduction to some of the techniques used when designing a drainage system but should be used in conjunction with Section 6 which presents more specific details relating to BMPs. The BMPs described in Section 6 should be applied when designing the drainage system.

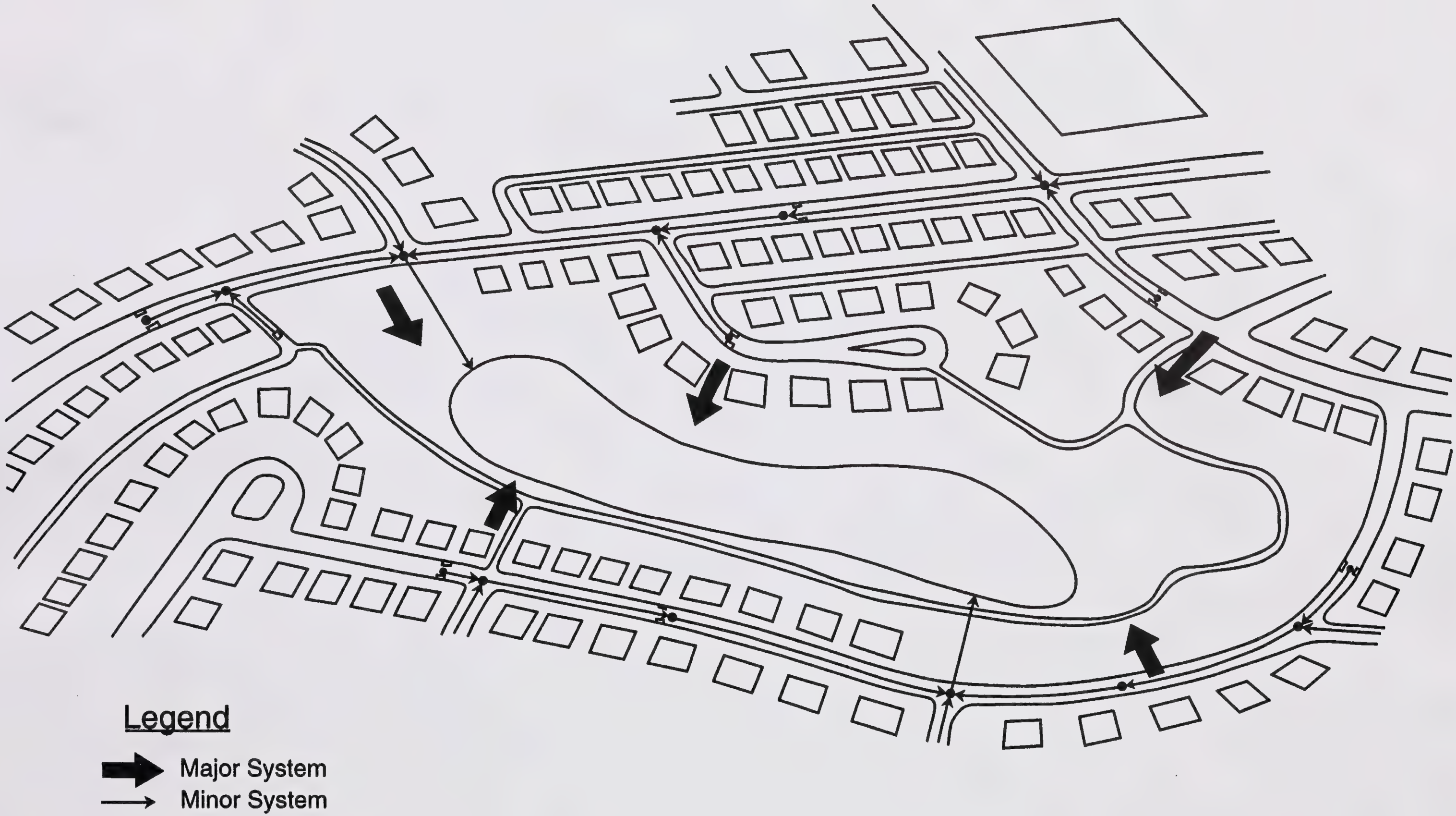


Figure 3.1
Major/Minor Systems

3.1.1 The Minor System

The minor system consists of those drainage works that transport flows from minor rainstorms quickly and efficiently from a catchment, providing convenience drainage.

The components of the minor system include:

- Roof leaders,
- Foundation drains (occasionally),
- Lot drainage,
- Gutters,
- Catchbasins, inlets, and leads,
- Underground pipe systems,
- Manholes, junctions, and outfalls,
- Storage facilities,
- Outfall channels,
- Erosion protection and energy dissipators, and
- Receiving waters.

The division of facilities into minor and major system components is not precise; some components, roof leaders for example, carry flow during both minor and major events. Such components are listed as part of both systems since they must be considered in the design of each.

3.1.2 The Major System

The major system consists of those drainage routes that transport flow during major storm events. Ignoring the major drainage system can result in potentially serious flooding of property during major rainfall events. The most common age system is property damage by overland flows. Unplanned major system runoff can also create hazards to life as it flows and ponds at locations where residents, particularly young children, are unused to it. Open manholes whose covers have been lifted off by surcharged storm sewers pose another risk, as do roads covered by water. Serious accidents can occur when drivers suddenly come upon flooded underpasses and low points.

The level of analysis and the effort to design facilities to transport major system flows must balance the relatively infrequent occurrence of such events and the seriousness of the damage they cause. It is generally accepted, however, that the level of effort directed toward planning for major drainage events in urban developments has been insufficient in the past.

The components of the major system include:

- Roof leaders,
- Lot drainage,
- Roads and gutters,

- Swales,
- Storage facilities,
- Outfall channels, and
- Receiving waters.

3.2 Level of Service

The objective of urban drainage systems is to provide a high degree of drainage service without causing unacceptable downstream impacts. This must be accomplished at a reasonable cost, leading to trade-offs between cost and level of service. The level of service provided by a system depends upon the interaction of the various system components. Problems caused by overloading the individual components vary between systems.

In the past, level of service has been established in a relatively simplistic manner, by designing underground pipes to carry the peak flow resulting from a 1-in-5-year rainfall event and spacing catchbasin inlets at suitable distances. The computational methods used to size the pipe systems could result in uneven capacities throughout the system; often the pipes in the upper portion had capacities above the design objective while pipes in the lower portion had capacities below the design objective. Since no consideration was given to the major events there was no consistency in the depths or flows in the major system or the extent to which flooding occurred.

The operation of each component of a drainage system must be examined to establish an even overall level of service at a minimum cost. Modern municipal drainage standards accomplish this through the provision of specific criteria for each drainage system component. Before establishing such criteria it is important to understand how the minor and major systems behave, both at and above their nominal design capacities.

In many municipalities, the capacity of the minor system is designed for the 1 in 5 year design storm while the major system is designed for the 1 in 100 design storm. These standards may vary depending on the willingness of the municipality to bear the higher cost versus the general acceptance, or lack thereof, of flooding problems. Some municipalities may apply higher standards for its major and minor system designs because of frequent flooding problems. They are best established by each municipality and specified in its drainage design manuals.

A design should consider factors such as safety margins versus cost savings when determining whether to reduce pipe size in the upper parts of the drainage system, whether to use inlet control devices, or whether to limit the number catchbasins at strategic locations on the street. Sometimes it may be desirable to have a higher margin of safety for a small additional cost. In general, storm sewer pipe shall be designed to convey the design flow when flowing full with the hydraulic grade line at the pipe crown during a 1 in 5 year design storm.

3.2.1 System Behaviour During Minor Storms

The minor system should quickly and efficiently remove the runoff from rainstorms below its design capacity. This runoff comes primarily from the impervious surfaces of a catchment: roofs, driveways, parking lots, and roads. Gutter flows are shallow and enter the piped system through catchbasins. The runoff then travels downstream through the pipe system to an outlet where it may be discharged either to a storage facility or directly to a receiving watercourse.

3.2.2 System Behaviour During Major Storms

At some point during a major event the capacity of the minor system is exceeded and overland flow conveyance begins. Depths of flow in gutters will be deep, so at points they will overflow to overland flow routes. These routes are usually swales leading either to storage facilities or directly to receiving waters. The minor system may also surcharge and flow under pressure at some locations. This should not be a problem unless weeping tiles or roof leaders are connected to the storm sewer system. During the 1 in 100 year design storm, surcharge in the minor system should not exceed basement levels and the flow depth on the street should not exceed, for example, more than 300 mm.

For systems with weeping-tile or roof-leader connections made inside the house, the amount of water entering the pipe system should be limited to what the system can transport without causing damage either to itself, to basements, or to downstream receiving waters. If inflows are not properly regulated, sewer systems flowing in a surcharged condition will result in the backup of water with the potential to cause basement flooding or structural damage. Water can enter basements through the connections or cracks or open joints; for well-sealed floors, a pressure head of as little as 0.15 m of water is sufficient to crack the concrete.

3.2.3 System Behaviour Above Capacity

A properly designed dual drainage system should be able to function effectively well beyond its design standard. The open channels and detention facilities that make up the major system will flow at depths and velocities greater than desirable, but apart from increased erosion in some areas there should be no appreciable damage. If structures are sited well above the design flood level, they should not be flooded except during the most extreme events.

3.3 Design Capacities

As noted in Section 3.2, the selection of nominal design capacities is dependent on the level of service desired, which may vary from one municipality to another. Economic analysis is rarely conducted to determine the optimal design event for drainage facilities; considerations and common practice related to setting design capacity requirements are outlined in the following sections.

3.3.1 Minor System Design Capacity

The establishment of capacity criteria for the minor system is largely a trade-off between cost and convenience in terms of level of service. From a convenience point of view it is desirable to set the standard on the high side, say the 10-year capacity. This, however, can become quite costly because of the increased pipe and excavation costs and possibly greater costs for downstream flow controls in comparison to a 5-year capacity.

In the larger urban centres in Alberta, the return period for minor system design most commonly used is the 1-in-5-year event. It has been argued that the 10-year event should be considered. As stated above, implementing either level of service is largely a balance between cost and convenience.

3.3.2 Major System Design Capacity

There is more variation in the selection of design standards for the major system because the concept of the major system is comparatively recent. The most commonly used major system design event is the 100-year event and is recommended for Alberta.

3.4 Drainage System Components

The previous sections have discussed the minor and major systems in terms of their overall behaviour during runoff events both at and above their design capacities. This section discusses the individual components of the systems and the considerations relevant to their design.

3.4.1 Roof Leaders

The most common means to accommodate roof drainage in Alberta is to direct it to ground that is graded away from the building. In a few systems, roof leaders are connected directly to the storm sewer. The latter practice can result in very large flows in the minor system and requires that the capacity of the system be increased accordingly. Roof-leader connection to the minor system is discouraged or prohibited in most jurisdictions. Most favour the discharge of roof leaders onto grassed areas, sometimes by means of "splash pads", well away from house walls. Discharging to pervious areas reduces the volume of runoff through infiltration and retards it through overland flow. Surface discharge of roof leaders is recommended by Alberta Environment.

3.4.2 Foundation Drains

Foundation drains, sometimes called "weepers" or weeping tiles, are the source of many urban drainage problems as they have the potential to cause basement flooding. This can occur when they are connected to either the storm or sanitary sewer systems.

On the Prairies, common practice in the past was to discharge foundation drainage from weeping-tile systems to the sanitary sewer. This practice relied on a small flow contribution from weeping tiles due to the low permeability of most soils native to the Prairies. If the

sanitary sewer system is a "tight" system (that is, minimal leakage into the system from poor lot drainage, manhole inflow, cross connections, and the illegal or inadvertent connection of roof drainage) then this method works well.

The practice of connecting foundation drainage to the sanitary sewer system is a problem where poor lot grading exists. Foundation drainage should be accommodated by discharge to groundwater through a gravel-packed recharge well (although this is not feasible in areas with impervious subsoils) or by use of a sump pump system.

In Alberta, it is recommended that each municipality carefully consider its own situation. If the local soils are permeable and the water table is high then a detailed evaluation of servicing by either connection to the storm sewer (with catch basin inlet controls and/or sewer backup valves), sump pumps, or three-pipe systems should be carried out.

3.4.3 Sump Pumps

Sump pumps can be used for foundation drainage, including existing systems that experience storm or sanitary sewer backup. The foundation drainage flows to a sump in the basement and is removed by the pump. Sump pumps shall not be discharged to sanitary sewer systems as the simultaneous discharge of several sump pumps can cause overloading. AEP recommends that sump pumps discharge to the ground, and that the ground be properly graded away from the house.

3.4.4 Lot Drainage

Proper lot drainage is an essential component of good stormwater management. It is important that the grade adjacent to new buildings be sufficient to allow for settlement of the fill and maintenance of positive drainage away from the structure. In many older areas, roof drainage runs out of the leaders only to flow back to the house and down the basement wall. This causes basement wetness and, in severe events, can cause backup of the sanitary sewer in areas where the foundation drains are connected to it. In some drainage systems, rear lot drainage is picked up by catchbasins along the rear lot lines. This is acceptable only under extenuating circumstances (due to high maintenance costs). The homeowner should also ensure that the ground elevation around building perimeters is well above the levels expected in the major system.

3.4.5 Catchbasins

The spacing and capacity of the catchbasins should be such that the widths and depths of flow in the gutters are acceptable during minor events. The relationship between the amount of water reaching an on-grade catchbasin and the amount that enters the catchbasin is called its "capture ratio". Proper spacing and capture ratio are important considerations in ensuring that the minor system provides the intended level of service.

Catchbasins should be constructed with a sump that will trap silt and other settleable debris that can pass through the grating. If a municipal storm sewer system is designed for self-

scouring velocities, and the catchbasin sumps are not cleaned regularly, it is recommended that sumps be excluded in the design of new systems.

3.4.6 Pipe Systems and Outfalls

There is an abundance of literature and standards dealing with the design of underground sewer pipe systems. If the amount of water entering such systems is properly regulated, the systems can be expected to perform as designed. The main difference between traditional and recent design practice is the consideration of inlet controls and the analysis of the behaviour of the system under major events.

When designing the pipe system, minor losses such as manholes, bends, drops, etc. should be considered as these can have a significant impact on the hydraulics in the system.

The analysis of erosion problems at outfalls is now quite commonplace, with erosion protection and energy dissipators constructed as necessary at outfalls.

3.4.7 Three-Pipe Systems

An alternative solution to the problem of foundation drainage is to employ a third pipe that carries only foundation drainage. This provides good and virtually fool-proof drainage to basements and allows the storm sewers to surcharge with virtually no consequences. In systems where weeping tiles are normally connected to the storm sewer, the three-pipe system may allow a 2-year capacity for the storm sewer and may reduce the cost of storm sewer servicing enough to justify the cost of the third pipe. Support for such a system is by no means unanimous, although it has been successfully applied in some areas and does provide reliable protection to basements. However, as this system requires another lead to the house, there is the possibility of cross-connection between the sanitary sewers and foundation drains.

3.4.8 Roads

During the peak period of a 100-year event, flows will likely fill local roads entirely; flow depths of no more than 0.30 m at the gutter are desirable. Standing water at low points should not exceed 0.50 m or extend to adjacent buildings. For arterial roads, the depths of flow should be less; typical criteria are that two lanes of traffic remain open and that the depth of flow be not greater than 0.05 m where major drainage flows cross arterials. No buildings should be allowed in the area flooded by the major event unless they have been specially designed with flood-proofing techniques to withstand flood waters.

3.4.9 Gutters and Swales

Gutters convey flow to the catchbasins during minor rainfall events. During these events the depths of flow are usually small and of little consequence. During major events, the much higher flows are conveyed in the gutters and in overflow swales. In these instances, the velocities and depths of flow should be examined more carefully.

Velocities in overland channels should be minimized. The force of moving water on objects in its path increases with the square of velocity. Table 3.1 lists approximate flow depths that a child (20 kg) would be able to withstand while standing in a concrete bottom channel or gutter flowing at the selected velocities. If the public has access to the flow route, these combinations of gutter velocity and flow depth should not be exceeded in open-channel design.

Table 3-1
Permissible Depths for Submerged Objects

Water Velocity (m/s)	**Permissible Depth (m)**
0.5	0.80
1.0	0.32
2.0	0.21
3.0	0.09

Note: Based on a 20-kg child and concrete-lined channels. Larger persons may be able to withstand deeper flows.

3.4.10 Receiving Waters

Consideration of the impacts of stormwater discharges on receiving waters should be implicit in modern stormwater management, with erosion, flooding, and water quality being the main concerns as discussed in Section 4.2. It is important to recognize that receiving waters form a part of the drainage system, and that the consideration of drainage does not end at the boundary of the development under design.

3.4.11 Super-pipes

Super-pipes (discussed in more detail in Section 3.5.2) are sewers that have been oversized to provide storage for runoff control. They are occasionally used in areas of existing development where trunk sewers are overloaded and the cost of land is high.

3.4.12 Outfall Channels

In many cases urban development does not extend to a receiving water body. In such cases outfall channels or ditches are the most economical means of conveying stormwater to the receiving water body. The use of open channels within the urban developments has generally been avoided due to safety, poor aesthetics, or high maintenance costs. Higher land values also reduce the economic advantages of channels within urbanized lands. There are some potential uses for open channels in stormwater management for urban areas in association with dry ponds as discussed in the next section.

3.5 Runoff Control

The need for runoff control is discussed in Section 4.2. Urban land developments greatly increase the volume and rate of runoff, mainly as a result of the large areas of impervious surfaces that they contain in the form of roofs, driveways, parking lots, and roads. The increased rate of runoff can usually be controlled by means of stormwater storage facilities that temporarily store the excess runoff and release it at a controlled rate. Normally, little can be done about the increased volume of runoff except in those few areas where infiltration or evaporation facilities are feasible.

The forms of runoff control include:

- Wet ponds,
- Dry ponds,
- Parking lot storage,
- Rooftop storage,
- Super-pipes,
- Catchbasin inlet controls,
- Infiltration areas,
- Soak-away pits,
- Cisterns,
- Evaporation areas/ponds,
- Splash pads to pervious areas, and
- Regulations limiting impervious areas.

3.5.1 Detention Ponds

As detention ponds are the most commonly used form of runoff control, a discussion of design considerations for these facilities is relevant. Detention ponds are also further identified as being either wet or dry ponds. Wet ponds are popular in many areas largely because they provide an aesthetic and recreational amenity.

3.5.1.1 Design Objectives and Effectiveness

Most ponds are designed to control runoff for a range of storms to meet erosion or flood control objectives. More recently, ponds have been considered for water quality improvement. Identifying the objective of the facility is the most important and sometimes most neglected stage of the design process. Establishing objectives normally means a review of the characteristics of the receiving water.

A common objective for a stormwater pond has been the matching of post-development to pre-development peak flows. This simple matching of pre- and post-development peak flows is not always an appropriate objective. Even if peaks are properly matched, the duration of flow under post-development conditions will be many times longer than for pre-development conditions. Events that previously caused only minor erosion as the peak

quickly passed through the system in the pre-development condition can cause extensive erosion damage after development. Also, the season of peak discharge may be changed. In many watersheds the maximum annual flows are caused by snowmelt. Urban development in part of the watershed may result in several periods of high flow during a year, one in the spring during the snowmelt and several others as summer storms occur over the urban areas. This can cause serious problems for farmers using low-lying land adjacent to watercourses.

In large urbanizing areas the effects of many ponds on downstream flood peaks also need to be considered at a watershed drainage plan scale. Normally, peak flows from different areas of a watershed do not coincide at a downstream point due to different travel times and variations in the rainfall distribution over a larger watershed. With a large number of storage facilities on a watershed discharging their peak flows for long durations, the peaks will become concurrent and directly additive. This has the potential for large increases in downstream flow.

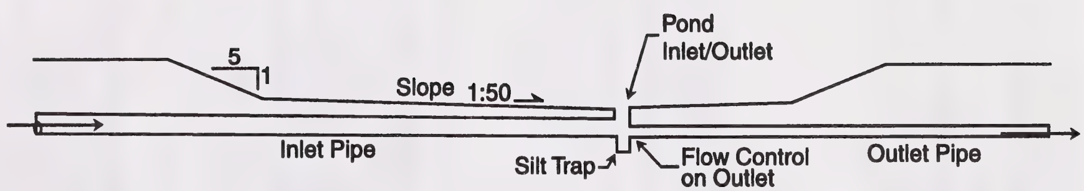
3.5.1.2 Wet Ponds

Wet ponds are usually built for storage of stormwater runoff remove pollutants and to flatten and spread the inflow hydrograph, thus lowering peak discharges. Attenuation is provided by storing the runoff peak flow. Removal of pollutants is accomplished by a number of physical, chemical, and biological processes such as sedimentation, flocculation, and metabolism by microorganisms and aquatic plants. Wet ponds are also provided to enhance the value of adjacent properties fronting on to the resulting lake. They are designed to be aesthetically pleasing, with curving shapes and even islands.

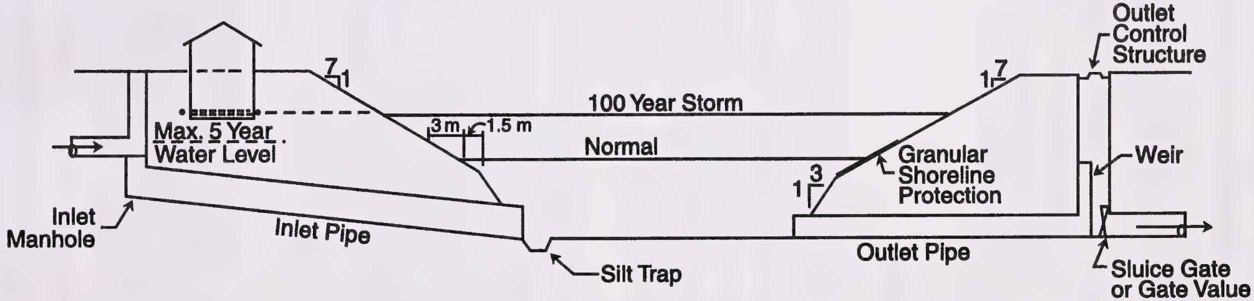
Outlet facilities for ponds can consist of a concrete weir, a berm with culverts at several levels, a mid-pond draw-off, or any one of a variety of other outlet structures. Sides lopes are typically grassed. However rip rap or gabion erosion protection shall be used where erosive wave action is a concern. Various edge treatments are also used to minimize onshore weed growth and maintain aesthetics. The pond side slopes are normally kept flat, typically between 5:1 (H:V) to 7:1 (H:V), to reduce the risk of slipping on wet grass and falling into the water. There is a trade-off involved here, however, as overly flat side slopes require more extensive edge treatment to accommodate movement of the water's edge during small rain events or extended dry periods.

Inlet pipes may enter a pond at different levels. In some areas the prime concerns are the avoidance of the effects of submergence on pipe hydraulics and siltation, while others feel that the aesthetics of the pond are improved if pipes have their inverts below the normal water level. For Alberta the pond inlet pipes should be submerged with their crowns below the anticipated ice level. Large-diameter inlet pipes may need special considerations (for example, multiple-pipe inlets). A manhole should be located as near to this inlet as possible, but at a location where the ground elevation is above the design high water level. Only the inlet pipe should be submerged; all other pipes upstream should flow without backwater effects during the minor system design events (Figure 3-2).

Water quality aspects of wet ponds are discussed in Section 6.



DRY STORMWATER DETENTION POND



WET STORMWATER DETENTION POND

Figure 3.2
Typical Pond Configurations

3.5.1.3 Dry Ponds

Dry ponds that only contain water during runoff events and for a short time after are gaining in popularity. While many of the design considerations for dry ponds are similar to those for wet ponds there are important differences.

The primary reason for using dry ponds is to maximize land use through dual usage of land dedicated for recreational uses. Large grassed areas such as football fields and ball diamonds can provide large storage capacities at relatively shallow depths. In some cases, municipalities have employed dry ponds to avoid potential water quality and maintenance problems associated with wet ponds. However, post-runoff-event cleanup and time lost for recreational use should be considered in evaluating the suitability of dry ponds. In addition to removal of silt and debris, restoration of minor damage to landscaping and facilities may be required. Debris management is important as poor management can have negative effects on pond water quality. Thus, dry ponds are often not suitable if normal rainfall events would cause frequent inundation.

Siting requirements for dry ponds tend to be less restrictive than those for wet ponds. In particular, the aesthetic impacts of size and shape are rarely significant. In some cases it may be possible to retrofit dry ponds into open space areas in existing developments. The dry pond storage concept can also be applied in a very linear form, such as a floodplain, or a natural or manmade channel used for stormwater drainage. By designing hydraulic constrictions into the channel, discharge rates can be regulated forcing temporary storage during major runoff events. This approach is the basis of the blue-green concept of integrating stormwater facilities with linear parks. Also, dry ponds are normally offline facilities that are only inundated when the sewer system capacity is exceeded.

Since dry ponds are designed to be dry and useful for active or passive recreation, adequate drainage is essential. Slopes will often be as flat as possible to maximize storage in the pond without resulting in excessive depths at the outlet. Bottom slopes should be at least one percent to facilitate lateral drainage. French drains or shallow drainage pipes can be used to prevent flatter sections from remaining wet for prolonged periods of time.

In all areas of the pond, slopes should be flat enough to facilitate grass-cutting and reduce the risk of slipping into the pond when it contains water. It is particularly important to have adequate signage in dry ponds warning against the dangers of sudden flooding. Painted markers at several locations showing the maximum water levels will give some idea of the possible extent of flooding.

3.5.1.4 Safety

The use of stormwater management facilities poses new hazards in the urban environment. As with any relatively new concept or product, it can be difficult to identify what constitutes an acceptable level of risk and good engineering practice. This is changing as stormwater facilities become more common features of urban development.

Each component of the drainage system poses its own risks. Children are naturally attracted to water bodies of all types; this must be considered in the design of stormwater management facilities. Particular care must be taken to avoid the creation of hidden hazards. A number of safety related considerations are:

- Ponds present an obvious hazard to non-swimmers and skaters Signs shall be posted that clearly indicate the nature of the hazards and that prohibit the inappropriate activities.

- Outfall structures and energy dissipators also present serious risks and may require fencing to restrict access. All exposed outfalls should be grated to prevent entry.

- Pond side slopes should be minimized to prevent people from slipping in; side slopes of 5:1 (H:V) to 7:1 (H:V) up to the 100-year level are recommended.

- Fencing of stormwater facilities must be considered carefully in each case. Children are expert at defeating fences which may only serve to hamper rescue efforts. Not placing fences may, however, be considered an invitation for children to use a facility and pose legal problems for the owner.

- Dry ponds pose an unusual hazard in that they are normally dry areas which may rapidly fill with water during extreme events. Children used to playing in such areas may not be aware of the hazard they present when full. Owners should consider being in attendance at dry ponds during the period in which they contain water.

3.5.1.5 Climatic Considerations

Alberta's climate is continental and comparatively dry. Winters are severe, and many areas of the province are unique. This is particularly true in the south, which experiences many freeze-thaw cycles as a result of Chinook winds. Summers are characterized in most areas by large variations in temperature throughout the day and high evaporation as a result of winds, low humidity and long hours of sunshine. In the southeast part of the province, annual precipitation is very low and the precipitation deficiency is high. Low precipitation and high evaporation may result in a need for makeup water to maintain normal pond operating levels and/or acceptable water quality in wet ponds. This need must be carefully evaluated, as it is not always desirable to use groundwater as a source of makeup water. Groundwater is a valuable resource in Alberta and makeup water is not considered a high priority use. Proper side slope treatment, such as lining a pond with a geosynthetic or clay liner, can usually overcome the need for using makeup water to maintain a normal water level if an aesthetic appearance can be maintained even when the water's surface is drawn down by 0.3 m.

Snow and ice may cause problems at pond outlet structures, and consideration must be given to avoiding blockage of inlet and outlet works. The orientation of the structure to the sun may be important in reducing ice buildup. Snow and ice may also block channels and cause

water to flood adjoining areas more frequently than intended. In urban areas, impervious surfaces tend to warm faster than pervious areas, so snowmelt may run off the development area to a pond or channel which is filled with snow. Unique climatic factors must be considered in each design. For example, after the stormwater control facilities are sized with design storms, detention ponds should be checked to make sure that they can also contain a 1-month or 2-week period of snowmelt within the freeboard. This would allow some protection against freeze-thaw cycles that may occur. Also, consideration should be given to the operational history of any similar works built in the area.

3.5.1.6 Sedimentation

Sedimentation must be considered in the design of both wet and dry ponds. Wet ponds must be capable of being drawn down to facilitate sediment removal or have facilities to pump down the water level. The means of sediment removal should be identified when the pond is constructed. Few data are available for predicting the required frequency of sediment removal; this can vary widely depending on the type of land use in the catchment and whether or not construction is still occurring in the area. Ponds are sometimes separated into two cells, with a smaller upper cell that traps most of the sediment. The cells are separated by a submerged berm. The upper area may incorporate access for sediment removal equipment and can have a concrete surface to facilitate scraping or dredging to a fixed level.

Sediment buildup in dry ponds may be more difficult to control because the bottom of the pond will be grassed, and even a comparatively small amount of sediment may be unacceptable aesthetically considering the recreational uses of the pond. One solution is to intercept at least the coarser sediment before it reaches the pond by trapping it in large "clean-out" catchbasins or larger sediment traps at the inlets to the pond. Another solution is to use the bypass concept in the pond design. Here, sediment deposits on the grass will occur only after major events (Figure 3-2).

3.5.1.7 Recreation and Aesthetics

Properly designed stormwater management facilities can provide recreational opportunities and enhance community aesthetics. While the water quality of urban ponds is generally not suitable for body-contact recreation, ice-skating may be possible in areas of the province where adequate ice thickness can be maintained for a sufficiently long period of time and the water level can be kept constant to prevent ice breakup from uplift. This type of use places a responsibility and an associated legal liability on the local authority for supervision and maintenance, together with the duty to ensure safe conditions.

The greatest secondary benefit from urban ponds may be their aesthetic appeal. Properly landscaped ponds can provide an attractive setting for a residential community. Once the engineering considerations have been established, landscape architects should work with the engineers to improve the shape and layout of the pond while maintaining its functional characteristics.

3.5.2 Other Runoff Control Alternatives

There are a number of other runoff control alternatives available for storm management. These are discussed in the following points, although not all methods will be applicable to Alberta.

3.5.2.1 Parking Lot and Rooftop Storage

Commercial and industrial developments normally contain parking lots, storage areas, and rooftops that have potential for stormwater storage. A problem with such facilities is that they are normally privately owned. The local authority may not be able to control the illicit removal of flow controls that are necessary to provide the ponding, in such areas, particularly on rooftops. In land developments where lots are sold to third parties, enforcing the implementation of the restricted outflow controls and storage on property is difficult. In commercial areas there may be resistance to ponding on parking lots because of the fear of inconveniencing shoppers caught away from their cars during a rainstorm. For these reasons privately owned storage facilities for stormwater management control may not be acceptable.

3.5.2.2 Super-pipes

Super-pipes are oversized pipes used to provide underground storage in the minor system. They are normally used only where ponds are infeasible since the cost of providing underground storage is very high. Such locations would be upstream ends of systems with high density development where open space is limited and land values are higher. Sedimentation can be a serious problem in super-pipes unless they contain low-flow channels, which add further expense. Additional costs should be included in operation and maintenance budgets to ensure the storage facility will be operational when needed. Super-pipes should also contain overflow sections to prevent upstream flooding should they become blocked.

3.5.2.3 Cisterns, Soak-away Pits and Infiltration Ponds

Cisterns are small covered tanks for storing water for a home or a farm; these are usually placed underground. Cisterns have the potential to provide a high degree of onsite stormwater storage, but their application is limited, probably because of lack of owner acceptability and municipal control. They may present a solution in areas with limited room for ponds and restricted drainage outlets.

Interception of roof drainage into cisterns can greatly reduce peak flow rates to the storm sewer. Cisterns can be designed to drain slowly to the storm sewer, or to soak-away pits (where soil conditions are suitable).

Few areas in Alberta have soils with sufficient permeability to facilitate the use of soak-away pits. Infiltration ponds are also limited in application in urban areas due to the need for large

areas with very permeable soils. On the scale of the individual lot, a cistern could drain to a pervious area such as a lawn, but only if the underlying soils are sufficiently permeable.

3.5.2.4 Evaporation Ponds

Evaporation ponds also require large areas to be functional. Evaporation and infiltration ponds may be viable in rural or semi-rural areas where land is available and soil conditions are suitable.

3.5.2.5 Limits on Imperviousness

Almost all runoff from minor storms and a large portion of the runoff from major storms comes from impervious surfaces. Limiting the impervious area will directly reduce the amount of runoff generated. However, since the degree of imperviousness is predetermined by the type of development most economically suitable for a site, it is generally impractical to prescribe limits on the portion of a site that may be covered with impervious surface.

Porous pavements are also a possible means of reducing runoff. This type of runoff control measure, while applicable to Alberta, has not been widely used.

4.0 Stormwater Quantity

4.1 General

Stormwater runoff involves the interaction of a number of phenomena. A rigorous analysis of the runoff resulting from a given rainfall event involves a large number of complex calculations. Prior to the general availability of computers, the time and labour required to carry out such calculations could rarely be justified. Thus, the use of simplified or approximate methods based on empirical relationships were commonplace and are still firmly entrenched in urban hydrology.

Many of the problems that have occurred in existing stormwater drainage systems have to some extent resulted from the use (or, more correctly, the misuse) of empirical methods. However, there is no guarantee that more sophisticated methods will eliminate future problems. They simply provide the ability to investigate the cause and effect relationships both in greater detail and with less effort. One important advantage of stormwater management analysis by computer models is that it provides a common basis of assessment for both the developer and the local authority.

Considering the complexity of the runoff process, any method of estimating runoff rates and runoff volumes should be applied with considerable caution. Such analysis requires both an understanding of the runoff process and the way a particular methodology portrays the process. This section provides some background for those involved in stormwater management analysis. The rainfall/runoff phenomenon is discussed, followed by a discussion of various methods for estimating runoff hydrographs and routing them through drainage systems. The section concludes with a description of some of the more commonly used computer models that are available in the public domain.

4.2 The Rainfall/Runoff Process

The amount and timing of runoff from a watershed is a function of several phenomena, which have varying degrees of importance depending on the nature of the system being modelled. The analysis of runoff processes includes the assessment of the precipitation event, interception and depression storage, evaporation, and infiltration. These latter items are called losses.

Interception storage is the amount of precipitation that can be stored by surface tension as it adheres to the vegetation in the watershed. This water later evaporates into the atmosphere. This may be 1 to 2 mm in forested areas and up to 4 mm in cropped land. It is a factor in the annual water budget and can be considered in rural runoff simulation. Interception storage, however, is not a factor during intense, short-duration rainfall events that are usually considered in urban runoff modelling.

Depression storage is water retained in puddles, ditches, and other depressions in the ground surface. This water may later evaporate into the atmosphere or infiltrate into the ground. For rural watersheds this factor is of considerable importance but cannot be quantified based on

land use or land form characteristics. As the nature of surfaces in an urban area is more regular and controlled, depression depths can be estimated. These are typically from 1 to 5 mm on paved surfaces and about 5 to 10 mm on grassed surfaces.

Evaporation and evapotranspiration of water to the atmosphere accounts for a considerable portion of the annual losses from surface water systems; however, it is of little significance in the analysis of peak runoff events in rural or urban watersheds, as evaporation rates are extremely low compared to peak precipitation rates. These water loss mechanisms are, however, factors in the post-event soil moisture depletion and the estimation of antecedent moisture conditions for subsequent events.

The infiltration of water through the soil surface is a significant factor in rural and urban watersheds. The soil infiltration rate is typically considered in terms of Horton's relationship:

$$F = F_c + (Fo - F_c)\, e^{-Kt} \qquad (1)$$

where: F is the infiltration capacity at time t,
Fo is the initial (dry) infiltration capacity,
Fe is the equilibrium (saturated) infiltration capacity,
t is the time since initial infiltration rate Fo, and
K is a the decay rate for infiltration.

The initial infiltration rate is significantly greater than the equilibrium capacity. Many rural and urban runoff models are based on this concept. Although urbanization greatly decreases the area of land available to infiltrate water, infiltration is still a significant loss component in an urban area. Very little of the pervious area contributes to the runoff during normal events, whereas for rare events the pervious area generates a significant amount of runoff. Equilibrium infiltration rates indicated in the literature are in the order of 10 to 20 mm/hr.

While most of Alberta's urban areas are located in the gently rolling terrain east of the Rocky Mountains and the foothills, there are several municipalities that are located along the eastern slopes. Stormwater management planning and design presents some unique challenges in these areas. From a hydrology standpoint, this is primarily due to difficulties in simply estimating the basic watershed inputs, such as precipitation and temperature, which can vary considerably with elevation. Forestation and vegetation of the watershed in higher altitudes can also have a significant impact on the hydrology, mainly in terms of stream base flow and runoff.

Because all or some of the watershed will be relatively steep, the runoff hydrographs will have high peaks and will be rapid, resulting in high velocities. These high velocities can result in significant debris being carried down from the upper part of the watershed.

Snow accumulation in the mountain region will affect the runoff characteristics. The higher elevation and the mature trees result in gradual snowmelts over a longer period of time. This results in a continuous base flow in the mountain streams throughout the year.

The flow component from snow melt could affect culvert designs at road crossings as well as land uses. A hydrologic model capable of analyzing runoff from rainfall and snow packs should be considered for mountain hydrology.

It can be seen from the above brief discussion that the selection of proper hydrologic parameters, although difficult, is paramount for evaluation of drainage systems in these areas.

4.3 Rainfall Considerations

The precipitation input for the generation of runoff from a watershed comprises either snowmelt, rainfall, or both. Snowmelt can be an important influence on the runoff from rural watersheds. Snowmelt runoff is often dealt with as part of a statistical hydrologic analysis of stream-gauging records. Although snowmelt/runoff simulation models are available and are used for large watersheds, they are not often used in assessing runoff conditions in smaller rural or urban watersheds.

Despite being a northern country with the implied abundance of snow and cold weather, the critical runoff events for the majority of Alberta's (and Canada's) cities are rainfall related. For urban areas, rainfall is the single most influential component in the generation of runoff. Because of the importance of rainfall, rainfall events are the subject of further discussion in the following sections.

4.3.1 Antecedent Moisture Conditions

The antecedent moisture condition (AMC) is a measure of the soil's current moisture content. It can be quantified by analyzing the amount of rain that has fallen in the hours, days, or weeks prior to a storm. Runoff coefficients, infiltration parameters, and other runoff model parameters can be adjusted by knowledge of the AMC. One application of this concept has been presented in the Soil Conservation Service's (SCS) National Engineering Handbook. It is used as a means to adjust the curve number (an SCS runoff parameter) for a given soil type based on the amount of rainfall that has occurred in the previous five days. An increase in the AMC means there is an increase in runoff potential from a watershed.

The SCS Handbook indicates that the AMC index is only a rough approximation of runoff potential as it does not include the effects of evapotranspiration and infiltration on watershed wetness. In estimating the AMC, their use of the prior 5-day precipitation does not address the greater importance of the rainfall immediately prior to the rainfall event or the effects of a large rainfall occurring prior to the 5-day period. A more refined antecedent precipitation assessment has been in practice for some time where soil moisture is assumed to decrease logarithmically with time during periods with no precipitation.

The Antecedent Precipitation Index, determined in this manner, is given by the equation:

$$\text{API} = S\, I_i\, K^i \quad (2)$$
$$i = 1$$

where: I_i is the precipitation on the i^{th} day prior the rainfall event, and
K is a recession constant (typically between 0.85 and 0.98).

There is little information available on relationships between antecedent moisture conditions and runoff model parameters. In the application of the SCS method, common practice is to use the most probable AMC condition. In Alberta this is typically AMC I (Table 4.1). For urban runoff models, no relationships are presented in the literature for pervious area infiltration rates as a function of AMC.

Table 4-1
Antecedent Moisture Data - Alberta

Location	Threshold No.			Antecedent Rainfall (mm)					Total (mm)
	Period	(mm)	Events	-1	-2	-3	-4	-5	
1-Hour Events:									
Edmonton	46-74	15.2	14	1.0	4.2	9.2	2.4	1.8	18.6
Calgary	50-74	15.2	11	3.4	4.2	0.1	2.0	1.3	11.0
Lethbridge	63-74	12.7	9	2.1	0.6	2.3	3.8	0.1	8.9
Vauxhall	58-74	12.7	10	3.9	0.2	0.5	0.3	0.7	5.6
Average	-	-	-	2.6	2.3	3.0	2.1	1.0	11.0
12-Hour Events:									
Edmonton	14-74	30.5	32.	8.1	1.5	0.9	0.8	1.8	13.1
Calgary	51-76	30.5	17	6.1	0.6	1.0	1.4	3.4	12.5
Lethbridge	61-76	30.5	18	10.5	2.0	3.5	3.0	1.5	20.5
Vauxhall	56-77	30.5	15	6.1	0.5	1.3	1.1	2.4	11.4
Average	-	-	-	7.7	1.2	1.7	1.6	2.3	14.4

Notes:
1. Based on data from AES, Environment Canada.
2. Soil Conservation Service 5-day AMC categories:

AMC	Dormant Season	Growing Season
I	< 13 mm	< 36 mm
II	13 to 28 mm	36 to 53 mm
III	> 28 mm	> 53 mm

4.3.2 Intensity Duration Frequency Curves

The amount (depth of rainfall and the rate (intensity) at which it falls are the most important aspects of a rainstorm. This information is well documented in intensity-duration-frequency curves (IDF curves), which relate the intensity of the rainfall to the duration of occurrence for various probabilities (Figure 4.1). These relationships have been based on short-duration rainfall data collected by Atmospheric Environment Services (AES) of Environment Canada, from locations they monitor throughout Canada. There are 28 locations in Alberta for which IDF curves have been developed (Table 4.2). Of these, sixteen are based on data from 20 or more years of record.

IDF curves are often expressed in functional form. Functional representation facilitates the precipitation data input to design storm generation and other stormwater management computer programs. The form of the most commonly used relationship for an intensity-duration relationship is:

$$i = A/(t + C)^B \tag{3}$$

where: i is the rainfall intensity,
t is the duration of the rainfall event, and
A, B, and C are regression constants.

AES has conducted statistical analysis for the monitoring stations in Alberta. They related the rainfall intensity to the duration (hours) using a simpler geometric regression analysis: $i = A/(t^B)$. This equation results in regressions that have poor standard error of estimate and should not be used. The tabulated data from AES should be used. Some representative values for the 5-year IDF curves are presented on Table 4.2.

Within any given year, many independent rainfall events occur. This is not addressed in IDF curve derivations by AES, which are based on single annual event statistics. With the AES method, the largest rainfall intensity for a specific duration for each year in the period of record is analyzed using the arithmetic Gumbel distribution. Where two or more severe rainfall events occur in the same year, the lesser events will be excluded from the analysis, even though they may exceed events in other years. Fortunately, the effect of this is not significant for the events rarer than the 5-year-return-period frequency. Studies have indicated that the intensity derived for a 5-year event in the traditional fashion (single annual event statistics) is really only about a 4.5-year-return-period event (a correction factor of about 1.04 is all that is required to compensate for this). As a result, there is only minor error in using single annual event statistics in normal stormwater management design.

Some authors propose the 2-year event for pipe design in dual drainage system analysis. When using the 2-year event, consideration should be given to the effect that partial duration series analyses would have on the 2-year IDF curve. The 2-year event estimated in the conventional manner underestimates the rainfall intensities by about 14 percent (a correction factor of 1.16 would compensate). This has the potential to cause a designer to underestimate the magnitude of 2-year-return rainfall intensities.

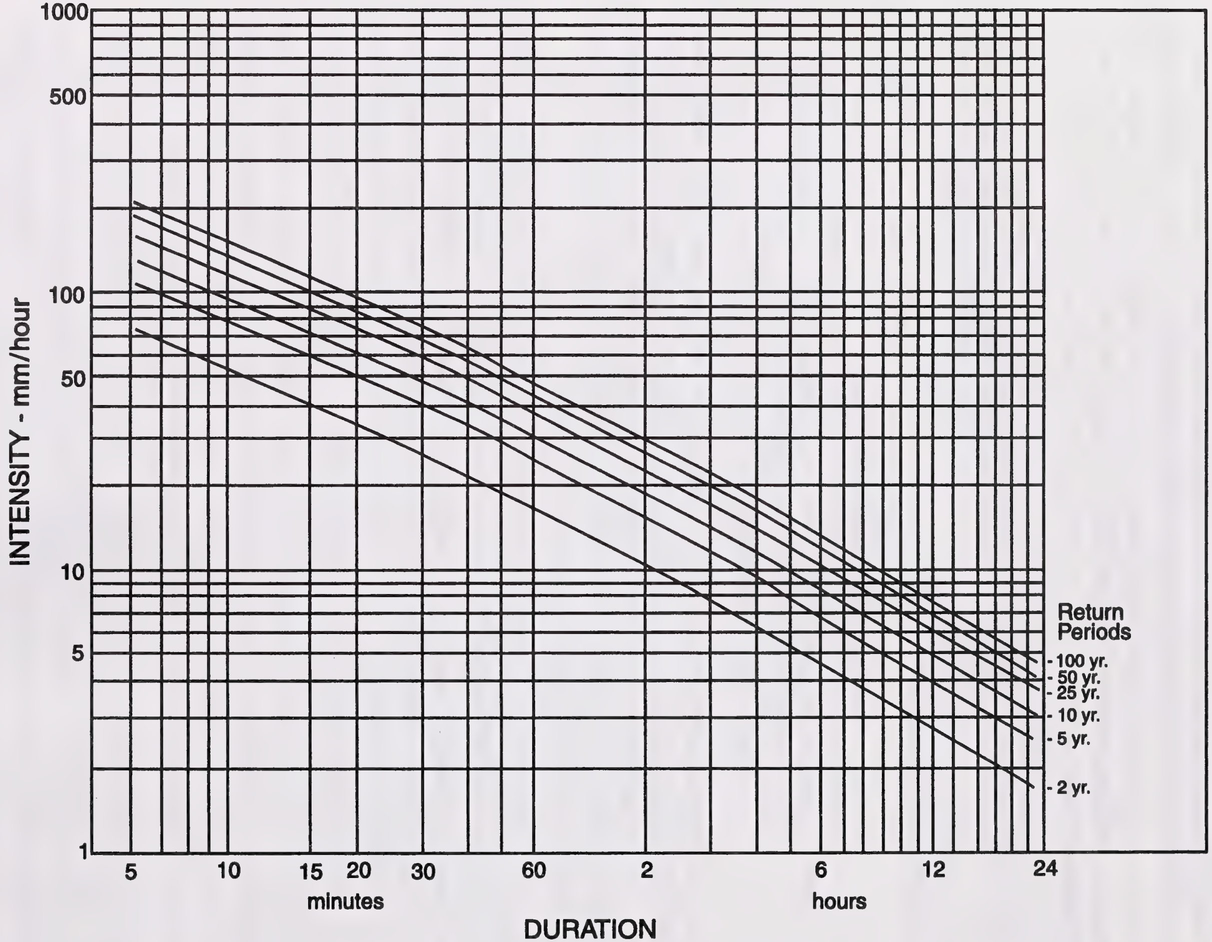


Figure 4.1
Typical I.D.F. Curve Format

Table 4-2
Atmospheric Environment Services Rain Gauge Stations
Rainfall IDF Data for Alberta

Station Name	Years of Record	Last Year of Record	5-Year Curve (mm/hr)			
			10 min	60 min	12 hour	24 hour
Beaverlodge CDA	24	1986	73.1	22.8	3.4	2.4
Brooks AHRC	19	1987	61.3	23.7	3.3	2.0
Calgary A	39	1990	67.8	19.1	3.5	2.1
Cold Lake A	25	1990	79.4	25.0	3.7	2.2
Edmonton Municipal A	56	1990	76.6	24.9	4.5	2.8
Edmonton International	28	1990	68.1	20.1	3.9	2.5
Edmonton Namao A	21	1986	68.0	19.8	4.0	2.7
Edson A	21	1990	80.9	21.9	3.7	2.5
Ellerslie	18	1986	66.2	22.3	4.0	2.6
Forestburg Plant Site	10	1982	96.9	26.6	3.3	2.2
Fort Chipewyan A	17	1986	49.5	15.6	3.1	2.0
Fort McMurray A	24	1990	55.6	17.4	3.7	2.2
Fort Vermilion	7	1972	53.3	20.9	3.5	1.8
Grande Prairie A	16	1986	53.0	14.8	3.2	2.2
Jasper	27	1990	36.3	9.6	2.7	1.9
Lacombe CDA	20	1990	75.0	23.6	3.9	2.5
Lethbridge A	30	1990	91.1	27.7	4.2	2.5
Manyberries CDA	15	1986	54.2	15.0	2.8	1.8
Medicine Hat A	19	1990	71.1	18.9	3.5	2.3
Mildred Lake	10	1983	48.3	16.9	3.7	2.3
Peace River A	24	1990	53.9	18.3	2.7	1.7
Pincher Creek A	22	1990	52.5	16.8	4.1	2.5
Red Deer A	26	1990	68.8	18.5	4.1	2.6
Rocky Mountain House	24	1990	77.2	23.1	4.1	2.6
Slave Lake A	18	1990	63.0	21.6	4.0	2.6
Vauxhall CDA	28	1983	59.8	20.0	3.2	1.8
Vegreville CDA	16	1990	--	28.3	4.0	2.4
Watino	14	1986	65.0	24.4	3.4	2.3

Note: Data can be interpolated if plotted on log-log paper. For other return periods refer to AES.

4.3.3 Spatial Distribution of Rainstorms

The variation of rainfall over a large area has been recognized in watershed hydrology. It has been difficult to quantify in smaller urban drainage areas. It can be significant on the Prairies, where thundershowers may move rapidly and may be quite local in nature. The cities of Edmonton and Calgary collect rainfall data at several sites to form a basis for evaluating spatial effects. As most urban watersheds are relatively small, it is now acceptable to assume that a uniform rainfall distribution based on point recordings at a rain gauge will represent average conditions in the watershed.

Using rainfall data from a low-level station to represent a high-level watershed will underestimate conditions. This phenomenon must be addressed in watersheds where orographic effects are of significance.

4.3.4 Temporal Distribution of Rainstorms

The temporal distribution of rainfall is the variation, with time, of the rainfall intensity during a storm event. A uniform-intensity rainfall event is a necessary simplifying assumption for the application of the Rational Method. The data to determine this intensity are conveniently given in the form of the IDF curves discussed earlier.

In small urban drainage basins, the peak flow rates are very sensitive to the storm distribution. For proper hydrograph analysis of stormwater management systems a more realistic temporal distribution of rainfall is needed. Where system storage is significant (either in large drainage systems or those with detention facilities) the design storm configuration is not quite as important.

To provide a reasonable and consistent basis for analysis for urban runoff modelling, a 5-minute time step is recommended for small urban areas. This is commensurate with the resolution of AES data. It is also less than the time of concentration (tc) of the smallest sub-basin that would usually be under investigation in an urban runoff modelling exercise. Typically, one should select a time step about one half of the minimum tc value.

Design storms for a particular frequency event can be developed based on synthetic methods or on historical data. These are described in the following section.

Synthetic Design Storms (Hyetographs)

The most commonly used synthetic method for developing design storms has been the Chicago Method. This method distributes the rainfall indicated by an intensity-duration curve of a selected frequency. The Chicago Method was developed ignoring the likelihood of the short-duration and long-duration rainfall intensities being concurrent (that is, the high-intensity short-duration rains tend to be isolated cloudburst events). As a result, the procedure produces design storms that are too peaky. This has been supported by several investigations. It was found that the peak 5-minute rainfall intensities determined in the

synthetic hyetographs are much higher than observations of the peak intensity during recorded 1-hour storms.

This overestimation of the short-duration intensities is one of the reasons for considerably higher peak flow rates being determined by most computer simulation models. This aspect of the storm hyetograph (the ratio of the peak 5-minute intensity to the average intensity for the storm) has been termed the "Peakness Factor" (or PF value). From a review of Toronto airport rainfall data it has been found that the observed PF values (which ranged from 2.2 to 2.8) were much lower than that for the 5-year, 1-hour Chicago-type synthetic hyetograph (which had a PF value of 5.9). For a given storm, increasing the time step decreases the PF value and, hence, the peak flow rate. It has been found that a time step of 10 minutes used in developing the 5-year, 1-hour Chicago hyetograph resulted in a PF of 2.9 (similar to the historic PF values).

The Chicago method for design storm development was popular for a number of reasons:

- The procedure is relatively easy to apply,
- It contains the critical aspects of storm intensity for all sub-basins within a catchment, and
- It is conservative.

As indicated earlier, the latter aspect of this procedure has led to considerable criticism being levelled at it. The problem can be related to the PF value of the synthetic design storm (for example, in Edmonton the 5-year, 5-minute discretization storm has a PF of about 4.1; historical data for the Prairies indicate a value of about 2.7). Adjusting the rainfall intensity during the most intense part of the hyetograph will compensate for this. A uniform rainfall intensity during the peak rainfall period equal to the time of concentration of the smallest sub-basin (usually greater than or equal to 10 minutes) provides a means for reasonable simulation results. The Chicago hyetograph for the Edmonton area, adjusted in this manner, has a PF of 3.0 (Figure 4.2). Hyetographs constructed in this fashion are believed to be reasonable design storm configurations.

Historically Based Design Storms

Use of historical design storms has been common practice for some time. Several such storms have been proposed for use in the City of Edmonton. Some have argued strongly in favour of the use of historical storms indicating that a design storm is "a device for facilitating analysis at the expense of credibility". The use of historical storms has public-relations value as well. The public can relate better to a system designed to handle the 1990 rainfall event better than one that can "handle the 10-year event". As a result, the use of actual historical storm events will continue to find application, particularly in the instance of major system design events.

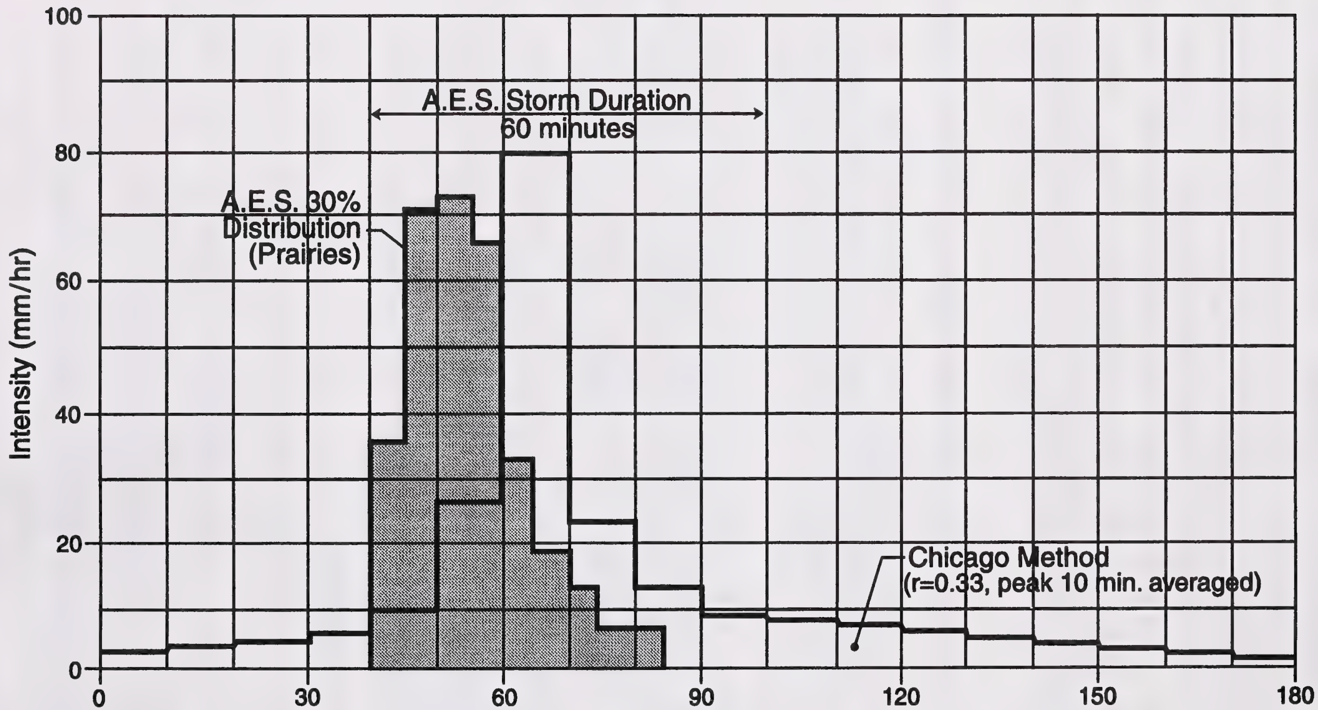


Figure 4.2
Design Storm Distributions
5 Year Return Period Event
Edmonton

Recent investigations have attempted to develop design storm distributions that are more realistic than the Chicago hyetograph based on historical storm event data. Research on the time distribution of rainfall has focused on the cumulative distribution of rainfall within the storm event. From a review of many storms, plots of cumulative rainfall (as a percentage) versus cumulative time (also, as a percentage) are made. From these, design storm distributions can be selected. This was done for 261 storms in Illinois where the first quartile (storms with the most rain occurring in the first 25 percent of total storm duration)were selected as the most representative of small urban areas. The average first quartile storm distribution is a built-in feature of the ILLUDAS computer model (Section 4.6.3).

There have been other such investigations conducted on design storm distributions. The most relevant for Alberta are the studies conducted by Atmospheric Environment Services (AES) of Environment Canada. Temporal distributions for 35 stations across Canada were developed for storms of 1 hour and 12 hours in duration. In subsequent work the distributions were given as percentile distributions. It was indicated that the 30-percentile distribution is most representative for use in simulation modelling. This conclusion was based on runoff simulations of each historical event and simulations of various design storm distributions and rainfall amounts obtained from IDF curves. Attempts have been made to minimize the discrepancy between the frequency distributions of the peak runoff rates from the historical and synthetic design storms.

A review of the AES work indicates:

> The main advantages of the AES design storms, which are used in urban drainage design, follow from the fact that they are based on actual Canadian data and are available on a nationwide basis. Shortcomings include the lack of guidance for the selection of other storm characteristics (td, D), the restriction to two durations, and a possible neglect of the variability of the temporal distributions with return period. Such shortcomings, however, could be overcome with a relatively small additional development effort.

In the above comment, td refers to the time step for storm discretization and D refers to the duration of the storm event.

4.3.5 Storm Duration

In small urban areas, it is important to identify the critical storm duration. For any given system element being designed, some experimentation with various storm durations should be conducted to determine the critical storm event. Previous studies indicate that the storm duration be greater than twice the basin's time of concentration. For most small (up to about 50 ha) urban areas, a storm duration of 1 hour is suitable.

For urban areas where detention storage is designed, longer-duration rainfall events are necessary to determine the critical volumes. The duration should be at least long enough that the peak storage volume is reached and the volume of stored water is in recession. The

designer must be able to determine that the detention facility will drain within a few days of the storm event. For this purpose, storms of 12 hours to 48 hours in duration are required. As these simulations are usually for major system events (for example, the 100-year event), the temporal distribution of the storm event is not as important.

For rural areas, experiences with rural runoff models (for example, HYMO) indicate that with the Chicago hyetograph, the peak flow rate increases as the storm duration increases. This effect is diminished by the time the storm duration reaches 24 hours. As a result, in rural areas, the storm simulation period should be 24 hours (or possibly more where storage is considered). The considerations for time-step discretization are similar to those for urban areas (that is, a storm duration greater than twice the basin's time of concentration).

For design storm durations of between 1 hour and 12 hours, the temporal distribution can be interpolated from the two AES distributions. The need for more analysis to identify the storm distributions for other durations has been identified. Until the results of such work are available, the use of the 30 percent AES Prairie distribution in this manner is acceptable.

In general, short duration and high intensity design storms such as Chicago distribution design storms are most suitable for analyzing urban runoff and designing sewer capacities as they tend to result in high runoff peaks. They are therefore more conservative. AES rainfall distribution curves for major cities across Canada can be obtained from AES. These rainfall curves can be used to distribute a rainfall hyetograph for analyzing stormwater runoff in urban cities. Long duration design storms such as the 12-hour SCS and AES design storm distributions are most suitable for analyzing runoff from rural areas and for sizing of detention ponds. In any case, the best approach is to simulate for all three types of design storms and use the one that is most conservative.

In addition to single event analysis using design storms, continuous simulation analysis can be used to assess the stormwater management system. Continuous simulation consists of running an entire meteorological record through a hydrologic/hydraulic model.

Single event analysis requires less computational effort, less data and design storms can be developed for an area, such as a city, and applied to many different projects. However, initial conditions such as antecedent moisture conditions in the soil have to be specified and temporal and areal distribution can have a significant impact on the results.

Continuous simulation, on the other hand, eliminates the uncertainties in determining an appropriate design storm. Continuous simulation also has the ability to account for antecedent conditions, thus eliminating the need to define the initial conditions for single event modelling. Continuous simulation does however require a lot of computational effort due to small time steps and long duration of modelling. It also requires a lot of data and model precision can be compromised by less detailed precipitation data.

Continuous simulation would be appropriate for evaluating the performance of a detention pond during a series of rainfall events. A typical average and wet year can be selected from past records for this purpose.

Single event simulations have the advantage of evaluating details of the drainage system that are usually too time consuming and expensive to carry out in the continuous mode. Continuous simulation is most useful for planning and optimization of preliminary designs. Single event simulation may then be used for detailed design and analyses. A combined approach of simulating a number of storm events and carrying out a frequency analysis may also be considered as an alternative approach.

A frequency analysis of historical rainfall events which generated flooding problems in the past can be performed. Historical critical rainfall events of known return frequency can provide a realistic assessment of performance of the system designed.

4.4 Runoff Estimation Methods

4.4.1 Rational Method

The Rational Method is based on an empirical formula relating the peak flow rate to the drainage area, the rainfall intensity, and a runoff coefficient. Undoubtedly the Rational Method is the most widely used method of predicting peak runoff rates for the design of urban drainage systems. Its popularity is a direct result of its apparent simplicity and ease of use. However, its simplicity is achieved by lumping all the complex variables involved in the runoff process into one coefficient. The Rational formula for metric units is:

$$Q = 0.0028\ C\ i\ A\ (m^3/s) \tag{4}$$

where: C is the runoff coefficient,
i is the rainfall intensity (mm/hr) for a storm of duration equal to t_c,
A is the effective (connected) area of the drainage basin (hectares), and
t_c is the time of concentration for the basin for the particular event (min).

The effect of this simplification has been widely discussed in the literature. For example, runoff for a 324-ha watershed computed by 23 designers, varied by 700 percent. This indicates that the underlying assumptions of the method and its limitations are still not widely understood after almost a hundred years of application.

The fundamental assumptions underlying the Rational Method are:

- The frequency of the runoff is equal to the frequency of the rainfall. This is not necessarily the case for any individual event, an important point when comparing computed values with measured values.

- The peak discharge at a point is a function of the average rainfall intensity over a duration equal to the time of concentration to the point in question. This assumes that the peak rate of runoff occurs at the point in time when the entire upstream basin is contributing, and the duration of the rainfall equals or exceeds the time of concentration.

- The rainfall is uniform over the basin and steady with time. Thus, a real distribution of rainfall or the tracking of storms across the basin cannot be directly accounted for.

Concerns regarding the proper application of the Rational Method include:

- Selection of runoff coefficients is highly subjective, and accounts for much of the variation in the results obtained by this method. Earlier discussion of the rainfall/runoff process shows that the coefficient C is not a constant, but varies with ground cover, soil characteristics, ground slope, depression storage, antecedent rainfall, rainfall intensity, and rainfall duration. Many publications list typical Rational C values but few qualify them as to storm type and frequency or soil conditions. Table 4-3 gives ranges of typical values for urban and rural conditions based on soil types and unfrozen ground conditions. Table 4-4 presents approximate values for specific return-period rainfall events.

  The potential error in estimating the runoff coefficient increases with the amount of pervious surface in the basin. This is particularly true for higher-return-period events as indicated in Note 2 in Table 4-3, which indicates the values should be increased for storms with return periods greater than 1 in 10 years.

- It is difficult to determine a realistic value for the time of concentration. This problem is more acute in rural or semi rural basins where there is extensive overland flow across pervious areas. Several methods for estimating the time of concentration are available. In urban areas it is common practice to assume an initial inlet time (ti) in the range of 10 to 20 minutes. However, this is a significantly wide range. For example, using the 1-in-5-year return IDF curve for the City of Edmonton (i10 = 83 mm/hr and i20 = 56 mm/hr), ti = 10 minutes yields a peak flow rate 50 percent higher than ti = 20 minutes.

  Downstream, the time of concentration (tc) is equal to the initial inlet time plus the time of travel (tt) through the conveyance system. As tc increases the effect of an error in ti diminishes, (e.g. at tc = 20 or 30 minutes the range of peak flow is reduced to 30 percent).

  The overall effect of using the incorrect inlet time would be that the system, in particular the upper reaches, could be designed for a storm with a lower or higher return period than was intended. It is imperative that the times of concentration used with the Rational Method are selected with considerable care for each branch of the system.

Table 4-3 Typical Urban Runoff Coefficients for 5- to 10-year Storms				
Description		**Runoff Coefficient**		
		Minimum	**Mean**	**Maximum**
Pavement	asphalt or concrete	0.70	0.83	0.95
Roofs		0.70	0.83	0.95
Business	downtown	0.70	0.83	0.95
	neighbourhood	0.50	0.60	0.70
Industrial	light	0.50	0.65	0.80
	heavy	0.60	0.75	0.90
Residential	single family urban	0.30	0.40	0.50
	multiple, detached	0.40	0.50	0.60
	multiple, attached	0.60	0.68	0.75
	suburban	0.25	0.33	0.40
Apartments		0.50	0.60	0.70
Parks, Cemeteries		0.10	0.18	0.25
Playgrounds		0.20	0.28	0.35
Railroad yards		0.20	0.28	0.35
Unimproved		0.10	0.20	0.30

Notes:

1. Values within the range given depend on the soil type if the watershed is significantly unpaved (sand is minimum, clay is maximum), and on the nature of the development.

2. For storms having return periods of more than 10 years, increase the listed values as follows, up to a maximum coefficient of 0.95:

   25 year - add 10 percent
   50 year - add 20 percent
   100 year - add 25 percent

3. The coefficients listed are for unfrozen ground. Taken from RTAC (1982).

Table 4-4 Selected Runoff Coefficients and Percent Impervious[1]					
Land Use or Surface Characteristics	**Percent Impervious**	**Frequency**			
		2	**5**	**10**	**100**
Business:					
Commercial Areas	95	.87	.87	.88	.89
Neighbourhood Areas	70	.60	.65	.70	.80
Residential:					
Single-Family	*	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2-Acre Lot or Larger	*	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.71	.72	.76	.82
Heavy Areas	90	.80	.80	.85	.90
Parks, Cemeteries	7	.10	.10	.35	.60
Playgrounds	13	.15	.25	.35	.65
Schools	50	.45	.50	.60	.70
Railroad Yard Areas	40	.40	.45	.50	.60
Undeveloped Areas	See "Lawns"				
Streets:					
Paved	100	.87	.88	.90	.93
Gravel	13	.15	.25	.35	.65
Drive and Walks		96	.87	.87	.88
Roofs	90	.80	.85	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.20
Lawns, Clayey Soil	0	.05	.10	.20	.40

Notes:
1. From Urban Storm Drainage Criteria Manual (Wright McLaughlin, 1969).
2. These Rational Formula coefficients may not be valid for large basins.

- Each reach of the system is designed for the peak flow from a unique theoretical rainfall event over the basin. Thus, the method gives no indication of how the system actually performs during a real event, especially an event that exceeds the design criteria. This is significant for the design of storage facilities, large trunk sewers, and in the design of relief works for inadequate drainage systems.

Although the Rational Method has major sources of errors, the potential impact of these errors in the design of small urban drainage systems can be reduced by careful selection of the appropriate parameters for the design event. Opinions as to the size of drainage system which may be designed by the Rational Method vary widely. The City of Edmonton allows the method to be applied for areas of up to 65 hectares, while other authorities suggest areas of between 200 ha down to a maximum pipe size of 450 mm. It is recommended that use of the Rational Method be limited to systems serving less than 50 ha.

The Rational Method has also been used to estimate storage requirements for stormwater impoundments. This is not a recommended practice as the potential for error is considerable. This is demonstrated by Figure 4-3 abstracted from Winnipeg's Drainage Criteria Manual.

The Rational Method is a simple and widely used method for the preliminary sizing of sewers using very few input parameters. A more accurate but more complex method of analyzing storm sewer systems can be carried out using computer models. Most computer models have a large number of input parameters and the model user should determine the most appropriate parameter values based on the user's manual and experience. Computer modelling is discussed in more detail later in Section 4.6

4.4.2 Isochrone Method

The Isochrone Method is a relatively simple way of estimating a runoff hydrograph for an urban catchment. The basis for constructing the hydrograph is a diagram of runoff-time-area and a rainfall hyetograph.

The time-area diagram is constructed by dividing the drainage basin into areas of equal time of travel to the point of reference, Figure 4-4. The time increment used should be the same as that of the design hyetograph. The hydrograph is computed in the manner shown on Figure 4-4, where i is the excess rainfall (after abstractions) at each time step. The method provides a hydrograph that reflects the effects of the rainfall distribution; this is more realistic than an assumption of a triangular hydrograph (a method sometimes used with the Rational Method).

The effects of the varying responses from pervious and impervious areas can be included in the method by developing time-area diagrams and excess rainfall hyetographs for each separately. The excess rainfall hyetographs for impervious areas are obtained by subtracting depression storage and allowing for the effects of surface routing. For the pervious-area hyetographs, additional abstractions must be deducted for infiltration using a relationship such as Horton's equation or published or measured values. The individual hydrographs are computed in the manner described above and are then added to give a total hydrograph.

This method produces a hydrograph based on a realistic storm pattern which reflects the effects of variations of rainfall abstraction during the storm. The time area diagrams are more easily and reliably computed where a conveyance system exists or has been designed by other methods. It is particularly useful for making preliminary estimates of stormwater storage requirements for urban drainage systems.

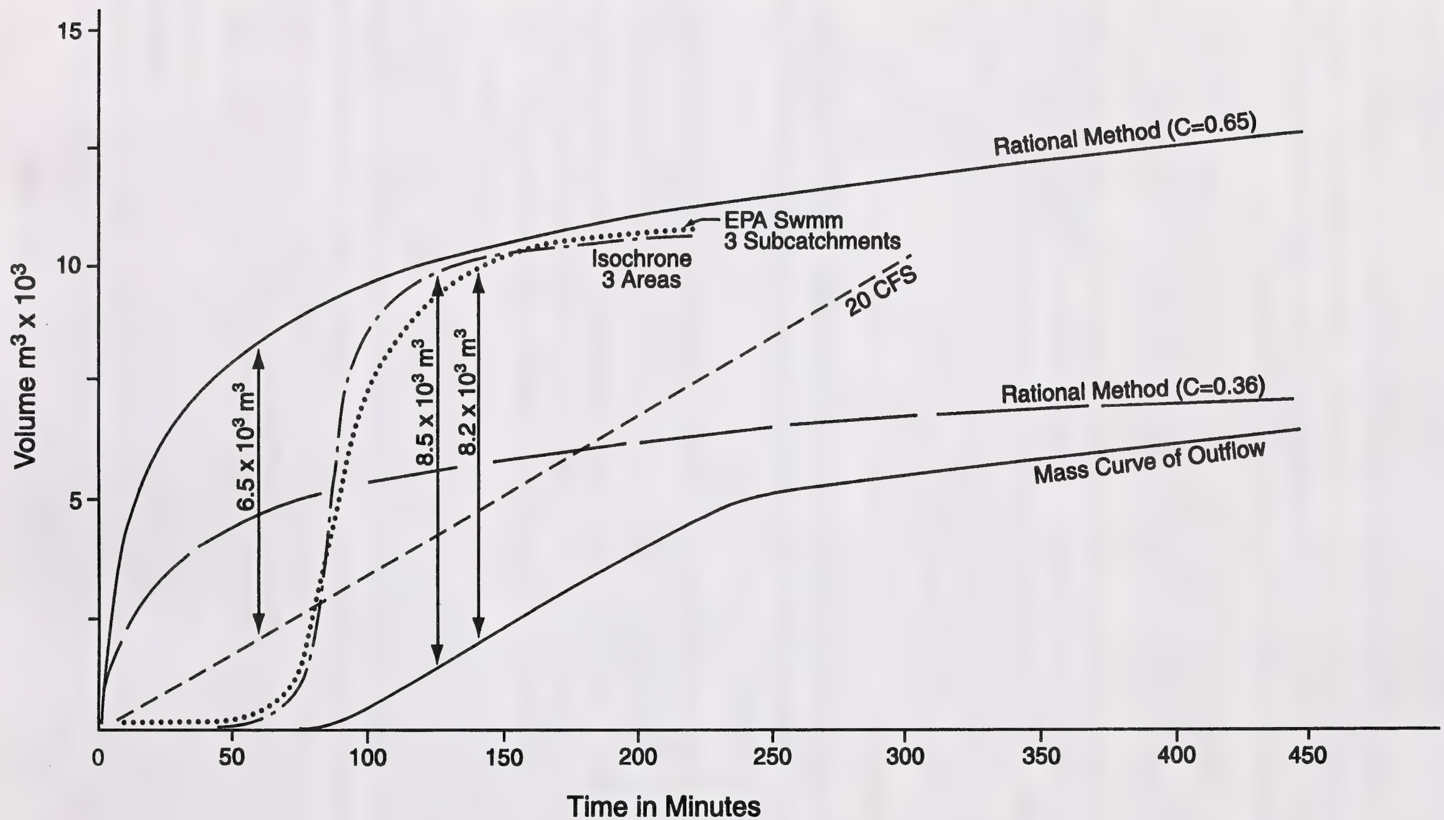
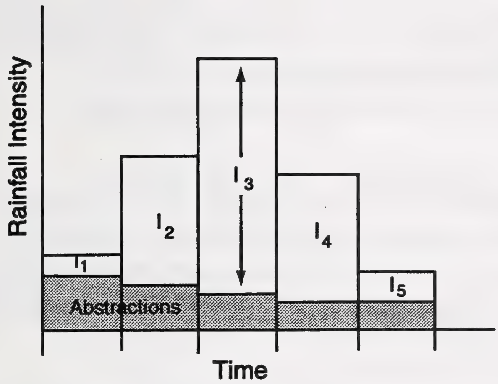
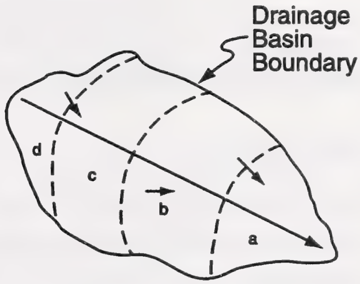


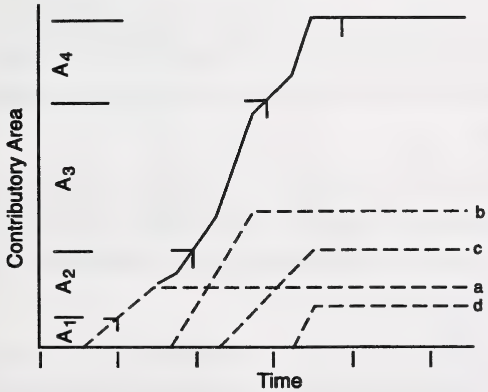
Figure 4.3
Storage by Rational Method



Rain Hyetograph



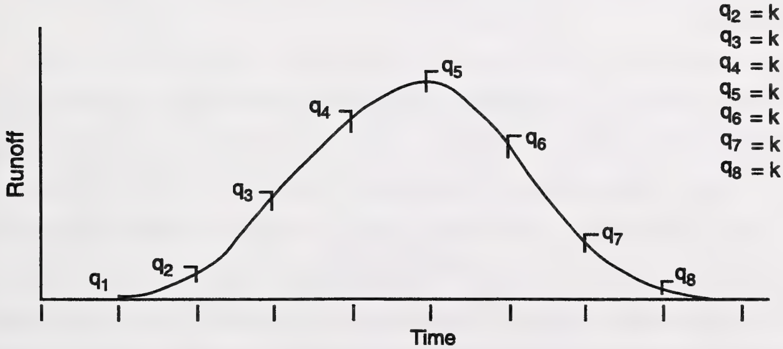
Drainage Basin



Time Area Diagram

Time Area Diagram

- Plot area contributing to outlet versus time for each sub-basin.
- Add sub-basin curves to get time-area-diag.
- Interpolate incremental areas (A_1 etc) for each time increment.



Runoff Hydrograph

Hydrograph

$$
\begin{aligned}
q_1 &= k\,(A_1 i_1) \\
q_2 &= k\,(A_1 i_2 + A_2 i_1) \\
q_3 &= k\,(A_1 i_3 + A_2 i_2 + A_3 i_1) \\
q_4 &= k\,(A_1 i_4 + A_2 i_3 + A_3 i_2 + A_4 i_1) \\
q_5 &= k\,(A_1 i_5 + A_2 i_4 + A_3 i_3 + A_4 i_2) \\
q_6 &= k\,(A_2 i_5 + A_3 i_4 + A_4 i_3) \\
q_7 &= k\,(A_3 i_5 + A_4 i_4) \\
q_8 &= k\,(A_4 i_5)
\end{aligned}
$$

Figure 4.4
Isochrone Method

The isochrone method does not involve complex calculations and can be carried out by hand for small areas where hydrographs are required at one or two locations. The calculations become time consuming for large areas and/or multiple hydrographs.

4.4.3 SCS Method

This method was originally developed by the Soil Conservation Service, U.S. Department of Agriculture for estimating the runoff from ungauged agricultural drainage basins. The method has been subsequently widely applied to all types of hydrology problems including urban drainage. The validity of such diverse applications has been questioned by numerous sources in particular with respect to urban hydrology.

In this method, the rainfall runoff relationship is expressed in the form of runoff curve numbers (CN). CN has little intrinsic meaning; it is a nonlinear transformation of a watershed storage parameter. In effect, CN relates total runoff to total rainfall for a wide variety of land uses for four hydrologic soil groups (A, B, C, and D) and three antecedent moisture conditions (AMC I, II and III). CN values for mixed land uses or soil types can be determined by simple weighting procedures.

The depth of runoff is computed by a simple equation using the total depth of rainfall and CN. CN ranges from zero (0) which will produce no runoff for any rainfall to 100 which produces 100-percent runoff for any rainfall. Runoff hydrographs are then generated by unit hydrograph methodology.

The SCS method can be used in hand calculations for small drainage basins. This is facilitated by charts or nomographs for urban applications published in the National Engineering Handbook and Urban Hydrology for Small Watersheds (TR55) published by the US Department of Agriculture.

4.4.4 Deterministic Methods

Deterministic methods quantify runoff from rainfall and/or snowmelt by simulating the effects of the various components of the process. This involves computing the runoff for a number of discrete time steps for the duration of the runoff event. Typically, surface detention storage is first abstracted from the rainfall followed by the abstraction of infiltration on pervious catchments. Infiltration is usually based on a relationship such as Horton's Equation, which relates the soil absorption and infiltration capacities with the current rainfall intensity. Next, the excess runoff is routed overland where additional infiltration may be deducted if the water flows over the pervious surfaces. At an appropriate point the pervious and impervious hydrographs are combined and may be further routed through channels to an inlet point.

Deterministic methods involve large amounts of calculation and are much more suited to computer modelling than to hand calculation. None of the existing computer models discussed in Section 4.6 are purely deterministic although a number of programs are classified as deterministic models. For practical reasons some processes and/or physical characteristics are lumped together and their impact is quantified empirically.

4.4.5 Snowmelt

There are two approaches to estimating snowmelt that are widely used, these being the degree-day method and the energy budget method.

The degree-day method simply relates snowmelt to mean daily temperature by a coefficient.

$$SM = C\,(T_a - T_b) \tag{5}$$

where: SM is the snowmelt in mm/hr,
C is a coefficient,
T_a is the mean daily air temperature, and
T_b is the base temperature above which snow melts.

The energy budget method relates melt rate to a number of atmospheric parameters.

$$M = M_{rs} + M_{rl} + M_{ce} + M_p + M_q \tag{6}$$

where: M is the snowmelt in mm/hr,
M_{rs} is snowmelt due to shortwave radiation,
M_{rl} is snowmelt due to long wave radiation,
M_{ce} is snowmelt due to condensation and convection,
M_p is snowmelt due to heat content of rain, and
M_q is snowmelt due to heat conduction at ground.

In practice, these equations are difficult to apply for two reasons. Most of the parameters involved have to be estimated, and the equations apply to a uniform snowpack (which is rarely the case). Also, unless recorded snow depth and snow density measurements are available (which is also rarely the case), these also have to be estimated.

For rural catchments the effect of snowmelt can generally be estimated from streamflow records using statistical methods of analysis.

4.5 Hydrograph Routing

4.5.1 Hydrologic Routing

The hydrologic approach to flood routing is based on the storage-depth and depth-discharge relationships of natural stream channels. Simply stated, the difference between the inflow and outflow rate at any time is equal to the rate of change of storage in the reach. This method assumes that the routed flow is changing slowly with time and that the dynamic effects of flow are negligible. These conditions apply to lakes and to some natural streams.

Numerous hydrologic flood routing methods have been developed based on the hydrologic routing concept. The Puls Method for reservoir routing and the Muskingum Method for routing hydrographs on rivers are well known and widely used. However, where streams

have steep slopes or where flow rates vary rapidly as in urban drainage basins, dynamic effects of flow may be pronounced. Other methods that account for the dynamic effects are noted in reference to various computer programs (Section 4.6).

4.5.2 Unit Hydrograph Methods

The unit hydrograph concept is widely used in hydrology. A unit hydrograph is defined as the hydrograph that would result from one inch of excess rainfall falling over the basin at a uniform rate during a specified period of time or duration. A unit hydrography is derived by analyzing recorded hydrographs from a gauged drainage basin and the associated rainfall. Various procedures are available for deriving a unit hydrograph. A further development of the method is the concept of instantaneous unit hydrographs, which provides a unit hydrograph that is applicable to all storm durations.

By definition, a unit hydrograph represents the routing effects of the physical characteristics of the drainage basin. Thus, a correlation can be made between unit hydrograph parameters (such as a peak discharge, lag time, and total base time) and basin characteristics including (area, basin slope, stream density within the basin, etc.). Using these derived relationships, a unit hydrograph can be transferred from a gauged basin to ungauged basin and be used to predict runoff.

There are numerous unit hydrograph methods, all using varying methods of derivation and differing relationships between hydrographs and basin characteristics, which reflect the hydrological data from which they were derived. These methods can be useful, but particularly in urban areas their validity for application should be verified.

4.5.3 Hydraulic Routing

Flows in storm sewers are generally unsteady and nonuniform when a pipe is not flowing full, and are subject to backwater effects from the downstream end of the pipe. Unsteady free-surface flow in sewers can be represented mathematically by the St. Venant or shallow-water wave equations.

Expressed in terms of velocity, the St. Venant equations are given by equations 7 and 8 below; expressed in terms of flow, they are given by equations 9 and 10 below.

$$\frac{\partial h}{\partial t} + D\,\frac{\partial V}{\partial x} + V\,\frac{\partial h}{\partial x} = 0 \tag{7}$$

$$\frac{1}{g}\,\frac{\partial V}{\partial x} + \frac{V}{g}\,\frac{\partial V}{\partial x} + \frac{\partial}{\partial x}\,(h\cos\theta) - S_o + S_f = 0 \tag{8}$$

dynamic wave

quasi-steady dynamic wave

diffusion wave

kinematic wave

$$\frac{1}{gA}\,\frac{\partial Q}{\partial t} + \frac{1}{gA}\,\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + \frac{\partial}{\partial x}\,(h\cos\theta) - S_o + S_f = 0 \tag{9}$$

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{10}$$

where: V is the flow velocity
Q is the discharge rate,
t is time,
x is the distance along the sewer,
g is the constant for acceleration due to gravity,
h is the flow depth above the invert (measured normal to x),
A is the flow cross sectional area normal to x,
D is the hydraulic depth (equal to the ratio of A to water surface width B),
θ is the angle between the sewer axis and horizontal plane,
S_o is sin θ, the sewer slope, and
S_f is the friction slope.

These equations provide a good approximate representation of unsteady sewer flows. Solutions of these equations for a sewer system are rather complicated, requiring the solution of numerous simultaneous equations or iterative numerical solutions. Some computer simulation models solve the complete equations but these models tend to be expensive to operate. This degree of sophistication is not always warranted with respect to cost and accuracy of simulation. Approximate solutions to the equations are obtained by eliminating various terms in the momentum equation (8 or 9) to produce simpler models as shown.

The quasi-steady dynamic wave approximation neglects only the local acceleration term, however it is less accurate than the diffusion wave approximation. The diffusion wave approximation neglects both of the first two inertial terms. However, the retention of the pressure term still permits attenuation of the hydrograph.

The Kinematic wave approximation neglects all but the slope terms in equation (8 or 9). With Sf estimated by Manning's formula or others, the flow is considered as instantaneously steady and uniform. This approximation is widely used in simulation models as it allows sewer routing to proceed pipe by pipe from the upstream end to the downstream end of a system. However, downstream backwater effects cannot be accounted for using Kinematic wave routing.

The Manning formula equation (11) is widely used to compute the steady state flow capacity for pipes and channels.

$$Q = A*R^{0.667}*S^{0.5}/n \quad (11)$$

where: A is the cross-sectional area,
R is the hydraulic radius,
S is the slope, and
n is the friction coefficient.

4.6 Computer Models

In recent years, the use of computer models for carrying out hydrologic and hydraulic analysis has increased rapidly. This trend is likely to continue in the future, particularly as the powerful personal computers now available are capable of using large and complex computer programs.

Numerous computer programs have been developed to model hydrologic and hydraulic systems. Most of these programs were developed for specific systems to provide specific information. Relatively few programs are designed as general application models which can be applied to a wide variety of problems in different locations. Such general application programs tend to require a large computer capacity and extensive input data. These programs also require a considerable level of effort to develop the expertise necessary for their proper use.

A number of programs that are useful in the planning and development of stormwater management systems are discussed below. The following are examples of programs that have been used extensively in North America. There are other models that can be used, such as MIDUSS, Wallingford, MOUSE, etc., and as long as the engineer can demonstrate that the model is appropriate and accurate as compared to the commonly accepted models, they can be applied.

4.6.1 Stormwater Management Model (SWMM)

This program was developed for the U.S. EPA and has been publicly available since 1973. SWMM is one of the most comprehensive modelling programs available for urban drainage system analysis. SWMM was developed to provide a common basis for evaluation of pollution abatement options for combined sewer overflows throughout the United States. To ensure that the model is applicable to a wide range of climatological and physiological conditions, the hydrologic and hydraulic routines are far more detailed than is normally required for water quality studies.

Because of the sophistication of its hydrologic and hydraulic routines, SWMM has been used extensively to analyze the operation of complex urban drainage systems. The program has received considerable support in Canada through the Ontario Ministry of Environment, the IMPSWM Group at the University of Ottawa, McMaster University, and a number of municipalities and consulting engineering organizations.

The SWMM package consists of six subprograms, called Blocks, which are controlled by a main program called the Executive Block. Although it was intended that the system should operate as one program through the use of overlays, many users find it convenient to operate individual blocks as separate programs. The function of each block is as follows:

Runoff Block

RUNOFF generates runoff hydrographs from a rainfall hyetograph and a physical description of the catchment areas. Simulated runoff is routed overland using the kinematic wave method. RUNOFF also generates pollutographs for the simulated rainfall event and user input pollutant loadings.

The program independently generates runoff for three hypothetical sheets representing the catchment surface.

Sheet 1 is an impervious surface with no depression storage. It therefore produces immediate runoff. Without this component no runoff would be simulated until the accumulated rainfall exceeds the total depression storage for the pervious area, an unrealistic condition.

Sheet 2 is an impervious surface from which all the impervious area depression storage specified by the user is abstracted before runoff occurs. The relative sizes of sheet 1 and sheet 2 are user-specified.

Sheet 3 is the pervious surface from which depression storage and infiltration are abstracted. Infiltration is computed by either the Horton equation or the Green Ampt equation as selected by the user. The program keeps track of both the accumulated rainfall and accumulated infiltration as well as the instantaneous rainfall and infiltration. It routes the excess rainfall across the pervious sheet and continues to infiltrate surface water in transit if there is excess infiltration capacity. Thus, if the rainfall hyetograph contains a very intense

rainfall time step followed by a low intensity time step, additional water will be abstracted during the second time step. It would be abstracted from that portion of the excess rainfall which did not flow off the pervious sheet during the preceding time step. While this represents a realistic simulation, it is a facility which can significantly effect results as discussed below.

The program adds the discharge from the three sheets together at the discharge node. This node can represent the inlet point to a sewer system, for subsequent pipe-routing by the TRANSPORT or EXTRAN BLOCKS, or the inlet point to a gutter or open channel through which RUNOFF can simulate additional routing.

All of the input variables for the RUNOFF BLOCK are deterministic, except for W, the width parameter. This parameter determines the shape of the catchment area. If W is relatively large with respect to the catchment area, a short overland flow route to the inlet will be simulated. Conversely, if a relatively small value of W is used, a long overland flow route to the inlet will be simulated. The effect of W is the same for all three flow sheets. The value of W can affect not only the shape of the simulated runoff hydrograph but can also affect the volume as runoff due to the infiltration simulation on sheet 3.

The SWMM manual suggests setting W equal to twice the length of gutter in the catchment as an initial estimate (which should be refined by calibration). The validity of this assumption will vary with the degree to which the drainage system is discretized and the physical layout of the catchment area. The impact of this parameter is often overlooked when calibrating a RUNOFF model.

Transport Block

TRANSPORT simulates free surface (open channel) flow of runoff through a drainage conveyance system of pipes or channels. Input data to TRANSPORT are hydrographs and pollutographs generated by the RUNOFF BLOCK (which are stored on a transfer file) and data describing the conveyance system.

The conveyance system is represented as a series of links and nodes typically representing the pipes and manholes of a sewer system. A number of other sewer elements including weirs, diversions (branched sewers), pump stations and storage facilities can be simulated as special types of nodes. A number of common pipe and channel cross-sections are available within the program, and unusual cross-sections can be user-defined.

In practical applications, it is not feasible to simulate individual pipes and manholes because of the large computer memory requirements this would entail. Typically, the nodes are used to represent junctions where significant changes in flow occur, conduit sizes change, or where the conduit slopes change. The Users Manual warns that there is an upper limit to the length of a link of about 1000 m required to maintain computational accuracy. There is also a lower limit to the length of a link. The link must be long enough so that water cannot flow from the upstream to the downstream end of the link (simulated conduit) in a shorter time than the computational time step specified for the simulation. If this situation occurs, water will be lost from the system.

TRANSPORT cannot simulate surcharged or pressurized flow in sewer systems. If the inflow at a point exceeds the 'full capacity' of the downstream conduit, the excess water is stored at the upstream node until the rate of flow recedes to the point where there is excess downstream capacity. The stored water is then returned to the system.
The flow routing routine used in TRANSPORT is quite sophisticated. The program approximates the solution of the St. Venant equations representing gradually varied, unsteady flow conditions in a conduit, using an explicit solution technique. Computations of flow conditions are made for all elements in the rondel at each time step, starting at the upstream ends and working progressively downstream.

Downstream effects are only approximated in TRANSPORT. This is not normally a problem except where pipes are steep enough to cause supercritical flows to occur. In such cases flows may be translated through the conduit without any routing. Also, where storage elements are used in the model, including super-pipes, the simulated output may imply false backwater conditions upstream of the storage element.

TRANSPORT has the facility to increase conduit sizes where simulated flows exceed the free flow capacity of the system. This facility can be used as a design aid for developing systems, particularly at the planning stage.

Extran Block

EXTRAN is a more sophisticated pipe routing program than the TRANSPORT BLOCK. It was specifically designed to model complex sewer systems. EXTRAN can simulate backwater conditions, looped pipes, flow reversals, and surcharged or pressure flow conditions. This enhanced ability to simulate complex systems is achieved by a methodology that solves the complete St. Venant equations.

Originally, EXTRAN contained water quality routing routines which have subsequently been removed as they were rarely used. The program has been used extensively for hydraulic analysis of complex sewer systems, particularly for projects involving flood relief of combined sewer systems. Poor documentation compared to other SWMM BLOCKS, combined with the level of effort required to develop the expertise to use the program, however, appears to have limited its use.

EXTRAN is far less user-tolerant than other SWMM BLOCKS due to the potential for mathematical instabilities to occur. Constraints on pipe lengths, slopes and specifications of special elements (weirs, flap gates, etc.) are more demanding. EXTRAN requires a very small computational time step, generally 5 to 20 seconds, resulting in a large number of calculation steps. Since it is impractical to output the results of each time step, numerical instabilities can propagate through the system between output cycles (which are generally at 5-minute intervals or longer). This makes it difficult to determine the source of such problems as the effects can appear both upstream and downstream of the source. One approach to minimize such problems is to develop and test large models in small segments.

The ability of EXTRAN to simulate surcharged flow was the principal feature that attracted initial users of the program. Unfortunately, early versions of the program were able to produce significant errors under heavily surcharged flow conditions. The solution technique uses the change in the in-system storage (physical pipe volume) over a time step to maintain continuity in the simulation process. To maintain numerical stability in the transition from in-pipe flow (relatively large storage) to surcharged flow (minimal storage at manholes), artificial storage is introduced to produce a smooth transition. In the early versions of the program, artificial storage was related to the system geometries and could become excessively large. This would result in excessive attenuation of the discharge hydrograph, and consistent underestimation of surcharge levels. This problem has apparently been controlled in recent versions of SWMM. However, users are cautioned to examine the results for heavily surcharged systems critically.

Storage/Treatment Block

The STORAGE/TREATMENT BLOCK simulates the impact of storage on the quality of stormwater effluent.

Receive Block

The RECEIVE BLOCK is used to simulate the impact of the quality of stormwater effluent on the receiving stream.

Combine Block

The COMBINE BLOCK is a utility routine which facilitates the modelling of systems too large to model as a single system. The COMBINE BLOCK can be used in a number of ways including collating data sets and/or combining data sets from one model run for input into another model.

4.6.2 SWMM - Derivatives

There are a number of modified versions of SWMM developed by various organizations. Of particular interest are the following:

4.6.2.1 CANSWMM - Canadian SWMM

This was an adaptation of the early version of SWMM for Canadian conditions developed for Ontario Ministry of Environment. Snowmelt routines were first incorporated in this version. Of particular interest in the development of this version was the application of the model to studies of water quality in receiving streams or water bodies.

4.6.2.2 DDSWMM

DDSWMM (Dual Drainage Storm Water Management Model) is a new release of the OTTSWWM model. In this model the RUNOFF BLOCK has been modified to allow simulation of a dual drainage system (minor and major) DDSWMM also takes advantage of the recent improvements to EXTRAN. Apart from its compatibility with the new generation of EXTRAN, DDSWMM has expanded on the size of the system that can be simulated, handling systems with up to 1,000 subareas, pipes and major system segments.

4.6.2.3 PCSWMM

This package is the latest version released by the EPA, SWMM4.2, Converted to run on IBM-PC and compatible micro-computers. Input and output have been modified to make it screen-orientated. The package includes an interactive preprocessor to facilitate the assembly of input data, including some error-checking capabilities, and a post-processor statistical package. The post-processor is designed to facilitate interpretation of continuous water quality simulations.

4.6.2.4 XP-SWMM

XP-SWMM is an enhanced version of SWMM coupled with the XP interface. The graphical EXPERT environment (XP) is a friendly, graphics-based environment which encompasses data entry, run-time graphics, and post-processing of results in graphical form. Drainage networks are either drawn on the screen over real-world topographical backgrounds or imported from a database. It has the ability to handle systems comprising pipes and open channels, rivers, loops, bifurcations, pumps, weirs, ponds, etc.

4.6.3 Hydrograph Volume Method (HVM)

In Canada, the HVM was first used in a number of stormwater management studies in Toronto in 1970. The model was subsequently used in Vancouver in 1974 as part of a combined sewer separation program in the City's West End. This model was the first commercially available stormwater model that could deal with conduit surcharging. However, the HVM model was the property of Dorsch Consult Limited of West Germany. In the initial decade of its usage, HVM could only be used by retaining Dorsch as a consultant. This restricted the usage and interest in the model considerably. As a result, SWMM with EXTRAN have enjoyed greater popularity. However, the HVM has been made publicly available on a time-share basis.

The HVM model comprises five programs which can be run together or separately. The interfacing of data between programs is accomplished by means of an Additional Data Tape. The Additional Data Tape is essentially a common formatted file for storing the output of the individually run programs (Partial Fill Curves, Model Storm, and Surface Runoff) prior to the execution of the Data Editing Program and finally the Sewer Network Flow Model. These programs are described briefly in the following sections:

Partial Fill Curves Program

The Partial Fill Curves Program determines several hydraulic elements for the various types of system conduits (including open channels) as a function of depth. The hydraulic elements are:

- Cross-sectional area
- Surface width,
- Hydraulic radius,
- Flow rate, and
- Flow velocity.

For flow depth greater than 50 percent, the program adjusts for the influence of air friction in partially full conduits.

Each typical geometric shape is analyzed and the computed attributes are stored on the Additional Data Tape. These hydraulic characteristics are referenced whenever that particular type of conduit is encountered in the Sewer Network Flow Model.

Model Storm Program

The Model Storm Program develops a synthetic design storm based on the Chicago method, originally developed by Keifer and Chu in 1957. Input to the model are the IDF curve parameters, the degree of advancement in peak rainfall intensity (which is the time to peak intensity divided by the total storm duration), the storm duration and the time step for the storm discretization. Output from the program is stored on the Additional Data Tape. In the newer metric version of HVM, this model is a subroutine of the Surface Runoff Model.

Surface Runoff Model

The Surface Runoff Model transforms the rainfall hyetograph (either a real event, a separately derived design storm, or the synthetic design storm from the Model Storm Program) into inflow hydrographs at a catchment's inlet point. For each surface type within the study area, the nature (that is, length, slope, roughness, and detention depth) of the roof, paved, and green areas are specified. The split between roof and paved areas and the domestic or dry weather flows are also identified here. The continuity and energy equations are solved to develop the specific runoff hydrographs for each surface type. Infiltration is accounted for using Horton's equation.

These typical surface-type hydrographs are stored on the Additional Data Tape for hydrograph construction in the Sewer Network Flow Model. When a sub-basin is identified in tile Sewer Network Flow Model, only the surface-type code, the area, and the percent imperviousness are needed.

Data Edit Program

The majority of the computer resource cost in using the HVM package is incurred by the Sewer Network Flow Model. The Data Edit Program is an error-checking facility that examines and flags any errors in the Additional Data Tape and the data file for the Sewer Network Flow Model.

Sewer Network Flow Model

The Sewer Network Flow Model simulates the dynamics of the stormwater management system during the storm event. The model simulates closed conduits, open channels, overflows, detention facilities, and diverging conduits. Up to 1100 conduits and special structures can be simulated in one run.

The basis of the HVM is an iterative solution to the St. Venant energy and continuity equations in finite difference form at each time step. The program has a significant advantage over SWMM's EXTRAN BLOCK in that the Courant condition does not have to be satisfied to achieve computational stability (a 5-minute time step is acceptable). The program fulfils its mathematical requirements by assuming that all of the upper ends of the system have no inflow and that all outfalls have an HGL which is a function of flow. A user can also specify the inflows and HGLs as functions of time by putting this information on the Additional Data Tape. In this way, hydrograph takeover from upstream systems or backwater conditions at outfalls can be simulated.

In its calculation procedure, HVM takes the surface runoff hydrograph for each conduit and forces the flow to enter the system along the length of the conduit. This is done regardless of the hydraulic grade line elevations that might occur in the conduit system. This representation is realistic as long as the extent of simulated conduit surcharging does not greatly exceed ground level (in HVM, ground level cannot be specified as an HGL constraint at a manhole). For events where the conduit system capacity is greatly exceeded and substantial surcharging above ground level is simulated, the hydraulics are not realistic. Peak flow rates for the conduits will be greater than those that will actually occur. This shortcoming does not preclude the use of the model for relief sewer planning (where conduit system capacity will be provided to accommodate a design event). However, use of HVM for major drainage system planning requires a considerable amount of judgement.

4.6.4 Illinois Urban Drainage Area Simulator (ILLUDAS)

The ILLUDAS model was developed as a tool to facilitate the design of urban drainage systems. It combines both hydrologic (runoff) computations and hydraulic (pipe routing) analysis in one model.

The hydrologic computations are based on the Isochrone method (reviewed in Section 4.4.2). Runoff hydrographs from impervious, supplementary impervious, and pervious areas are computed separately. The supplementary impervious hydrograph is added to the pervious

surface hydrograph prior to the computation of infiltration losses. The combined hydrograph is added to the impervious area hydrograph at the inlet point. Although infiltration is computed by the Horton equation, user options are limited to selecting one of the four Soil Conservation System hydrologic soil groups and one of four antecedent moisture conditions. Depression storage estimates for both pervious and impervious surfaces are user-defined.

A simple storage routing technique is used to simulate flow through pipe or channel sections in the drainage network. The technique uses storage discharge relationships computed by the planning formula and a simple storage routine formula. Complete hydrographs are routed through each reach in succession. Backwater conditions are not simulated in ILLUDAS.

The model temporarily stores runoff in excess of each pipe capacity at the upstream pipe node and can output nodal storage volumes. This feature is useful for determining preliminary storage requirements in the design of stormwater storage facilities.

4.6.5 Hydrologic Model (HYMO)

The HYMO program was developed by the U.S. Department of Agriculture in 1973 and is described as a problem-orientated computer language for modelling surface runoff and sediment yield. The program is designed to be highly interactive allowing the user to carry out a step-by-step analysis using a set of command words. The model can also be operated from an input file of commands and data. These commands allow the user to compute a runoff hydrograph, compute a rating curve for a channel reach, compute a travel time table for a channel reach, and route a hydrograph through a channel reach or a reservoir. Additional commands enable the user to store and retrieve hydrographs and rating tables, add hydrographs together, and print or plot output.

Rainfall input to the model is in the form of a mass curve instead of the more usual hyetograph format. The program can only retain six hydrographs in memory at a time, a minor limitation for large systems which can be overcome with careful sequencing of the analysis steps.

Channel routing is carried out using the Variable Storage Coefficient method with modifications to account for changing water surface slope during a flood. HYMO uses the storage-indication method to route floods through reservoirs. These routing capabilities are superior to those used in ILLUDAS based on studies carried out by the IMPSWM Group.

Hydrographs are generated using the SCS method to determine the rainfall excess and a unit hydrograph developed by the program authors. While this approach provides acceptable results for a wide variety of rural drainage systems, it does not produce accurate hydrographs for urban catchments.

The simplicity of this model in terms of both user modelling expertise and data requirements make it an attractive model for drainage planning purposes, and led the IMPSWM Group to develop additional procedures to model urban drainage components.

OTTHYMO/INTERHYMO

The OTTHYMO program contains all of the original HYMO commands plus three additional commands (URBHYD, NASHYD, and KINROUTE) developed by the IMPSWM Group.

The URBHYD routine is designed to produce results consistent with those produced by the RUNOFF BLOCK (SWMM) for large lumped catchment areas. URBHYD uses essentially the same hydrologic input data as RUNOFF. Where the URBHYD and RUNOFF routines essentially differ is in their hydrograph routing procedures. URBHYD uses unit hydrographs derived from a synthetic linear reservoir system concept to simulate the lag effect of overland flows.

Runoff hydrographs from pervious and impervious surfaces are computed separately using different reservoir systems.

The NASHYD routine gives the user the option of using either the NASH unit hydrograph or the Williams and Hann unit hydrograph originally used in HYMO. The NASH unit hydrograph produces a shorter recession limb for the output hydrograph which may be more appropriate for small rural watersheds. In addition, the user can specify the initial rainfall abstraction instead of using the fixed relationship in the SCS method in HYMO. The initial rainfall abstraction can be based on the Antecedent Precipitation Index (see section 4.3.1).

Rainfall input for OTTHYMO is in the form of a hyetograph, making it compatible with most other urban drainage models.

KINROUTE is a routine for routing flows through pipe systems based on the diffusive kinematic wave model. Validation of the model has indicated that results compared favourably with sophisticated dynamic models for free surface flow conditions. The entire hydrograph is routed through a pipe section before proceeding downstream. This is necessary to conform with the basic operation of the HYMO program. Backwater effects are not simulated.

INTERHYMO is a more recent version of OTTHYMO developed by Paul Wisner & Associates in 1989. INTERHYMO contains all of the capabilities of OTTHYMO but it is expanded further with new subroutines including derivation of design storms, quasi-continuous simulations, lag of rural and urban peak flows in determining the runoff hydrograph, shifting of hydrographs, calibration parameter file, modified areal distribution factor for meteorological data, and interface with EXTRAN.

4.6.6 Hydrologic Engineering Centre Programs

The Hydrologic Engineering Centre, a section of the Corps of Engineers, U.S. Army, has developed and continues to maintain a number of comprehensive programs for modelling hydrologic engineering problems. In particular the programs STORM, HEC-1 and HEC-2 may be useful in some stormwater management applications.

4.6.6.1 STORM

The STORM program is primarily a water quality model capable of continuous simulation of runoff events and pollutant loadings from hourly precipitation records. The hydrologic routines in the program are relatively simple. If the average daily temperature is below a specified threshold value, precipitation is accumulated in a snowpack. If the average daily temperature is above the threshold value precipitation is treated as rainfall and/or the residual snowpack is melted at a rate computed by the degree-day method.

Two methods are available for computing runoff. A coefficient method assumes a constant ratio of runoff to rainfall minus depression storage. Recovery of depression storage, from a specified maximum value, is computed continuously from an input average evaporation rate. This method is more suitable for highly impervious drainage areas than for pervious areas.

The second method is the SCS Curve Number Technique with provision to recover infiltration and detention storage capacity during dry periods. This method is more suitable for pervious drainage areas. The program will simulate dry weather flow for combined sewer systems. A triangular unit hydrograph concept is used for routing flows through the drainage basin.

The water quality routines allow STORM to simulate the accumulation of pollutants on the drainage basin during dry weather, the pollutant wash-off during runoff events and the impact of overflow diversions, in-system storage and treatment on the discharge of pollutants to a receiving water body.

As a hydrologic model, STORM is not suitable for simulating high-intensity short-duration rainfall events which typically control the design of many urban drainage systems. However, it has been used to search historical rainfall records to identify historical high-runoff events or high-runoff periods.

4.6.6.2 HEC-1

The HEC-1 program is described as a flood hydrograph package which can model runoff from precipitation on a complex, multi-catchbasin, multi-channel river basin. The program is limited to analyzing single events as there is no provision for recovery of rainfall abstraction rates during dry periods.

Snowmelt can be computed using either the degree-day method or the energy budget method. Rainfall losses (infiltration and depression storage) are computed by a function relating loss rates to rainfall intensity and accumulated loss. The program will compute a unit hydrograph or will apply a user-supplied unit hydrograph. Channel routing can be carried out using one of six optional hydrologic procedures (Modified Puls, Muskingum, Working R and D, Straddle Stagger, Tatum, or Multiple Storage).

An unusual and valuable feature of the program is that it will derive loss rate and unit hydrograph coefficients or routing coefficients from a recorded hydrograph. A procedure to use this facility to calibrate a system model is described in the program users' manual.

The program also has the facility to evaluate alternative scenarios in terms of floodplain damage-flow frequency relationships. Scenarios can include existing conditions, changes in land use, increased in-system storage, or improved channelization. Comparisons are made on the basis of estimated average annual damage costs.

4.6.6.3 HEC-2

The HEC-2 program compiles the water surface profile for river channels of any cross-section for either subcritical or supercritical flow conditions. The effects of bridges, culverts, weirs, and other hydraulic structures can be modelled.

Storage discharge curves for river reaches can be transferred to HEC-l for subsequent stream routing analyses. This program is widely used to compute backwater profiles in rivers and open channels and for floodplain delineation.

4.6.7 Model Calibration and Verification

Virtually all documentation and references for stormwater modelling programs emphasize the need for calibrating and verifying models developed for each system studied. Unfortunately, this is impractical in most cases due to a lack of measured flow data, especially for designing new urban drainage systems. Data are more likely to be available to calibrate rural watersheds, as a considerable number of small streams in Alberta are gauged.

Short-term flow monitoring programs are recommended for model calibration purposes in the analysis of existing urban drainage systems. This has been carried out in many municipalities. A successful flow monitoring program can be achieved with a good quality assurance/quality control (QA/QC) program. A QA/QC program may include some of the following procedures:

- Careful selection of a monitoring site to ensure that it is not affected by backwater, flow turbulence, or pipe sediments
- Calibration of the flow monitor in a laboratory environment to ensure the depth and velocity sensors are accurate
- Calibration of stage-discharge relationship and the Manning's roughness coefficient for the monitoring site using the flow monitor and a hand held velocity meter
- Robust equipment to withstand the harsh sewer environment (constant maintenance is required to clean the sensor probes)

- Frequent download of electronic data to review and check for questionable data deviations

- Carry out mass balance calculations for a network of flow monitors to confirm that there is flow data continuity

Long-term flow monitoring provides a more comprehensive flow database than does short-term monitoring. With a larger database, there are adequate opportunities for QA/QC and adjustments of flow monitors to improve the accuracy of the data collected. However, long term monitoring programs are more expensive. The City of Edmonton, for example, has an extensive flow and rainfall monitoring program which has been in operation for several years. Other municipalities should endeavour to implement long term rainfall/runoff monitoring. In time, such programs will provide a better insight into the hydrologic characteristics of their urban drainage systems and facilitate model calibration.

With little or no calibration data available, deterministic models such as SWMM and its derivatives have generally been acknowledged to be the best representation of the rainfall/runoff phenomena in urban systems. The user, however, must exercise a considerable degree of judgement in choosing parameter values and should examine the effect of each variable on the simulated results.

Irrespective of the sophistication of a computer model, it is possible to generate significantly different results for the same project with seemingly minor variations in the input data. It is therefore reasonable for local authorities to require those conducting model studies to justify the results obtained by their models. An indication of the sensitivity of the results should be given.

5.0 Stormwater Quality

5.1 Introduction

Historically, there has been a tendency to regard stormwater as a relatively minor pollution source - a nuisance rather than a real problem. Numerous studies since about 1970, however, have noted that there is significant pollution from stormwater. Urban runoff can have characteristics similar to raw sewage. It is usually high in suspended solids and organic matter that exert an oxygen demand in the receiving waters. It can contribute significant concentrations of toxic metals, salts, nutrients, oils and grease, bacteria, and other contaminants. Stormwater discharges to receiving waters can thus have significant impacts on potable water supply, aquatic habitat, recreation, agriculture, and aesthetics.

The natural environment has some ability to mitigate the impacts of pollution. Relatively small urban areas draining into large lakes or rivers are unlikely to cause significant effects. If, however, an outfall is located close to a beach, a domestic water supply intake or a biologically sensitive area, it may be a significant pollution source. Surface water quality objectives are the main factor to be considered when evaluating the significance of an urban stormwater discharge. However, the use made of the receiving stream is also an important consideration.

In evaluating stormwater discharges from an urban environment, the cumulative impacts of future development and the background substance loads in the receiving water should be considered. While one outfall may not pose a significant problem, it can set a precedent allowing the future construction of outfalls to the point where an undesirable situation results. It is likely that little can be done to correct the problem when it does become apparent, since retrofit techniques are often prohibitively expensive. This situation is no different from that associated with increased flows and flooding, and provides further support for the establishment of watershed drainage plans and master drainage plans for developing areas.

The presence of other sources of pollution must also be considered. If a lake or river is exhibiting signs of distress as a result of domestic, agricultural, or industrial pollution, it may be undesirable to introduce additional pollutant loads unless they are very small compared to the background concentrations.

Generally, urban stormwater is a controllable source of pollution. As a minimum, treatment in the form of sediment control should be encouraged where feasible. Proponents of stormwater drainage systems in Alberta are advised that treatment in addition to sediment control may be required in cases where water quality impacts on the receiving watercourse are of particular concern. Furthermore, this requirement may also lead to a regulated monitoring program in particularly sensitive watersheds.

Stormwater runoff can contain a wide variety of contaminants, often at concentrations substantially exceeding ambient surface water quality objectives. The chemical makeup of stormwater is primarily dependent on the land use within the catchment and the location of the urban area with respect to sources of atmospheric pollution such as major industries or other large developments. The concentrations and loadings of various contaminants in stormwater are directly related to the land use characteristics by the amount of impervious surface found within the catchment area. In general, the stormwater quality discharged from a catchment deteriorates and the volumes of runoff increase as the percent impervious area increases. Measured contaminant concentrations from a number of studies are shown in Table 5-1 as an indication of typical stormwater quality.

It is difficult to relate concentration and loading data from other studies to specific catchment areas. The normal practice is to relate anticipated runoff volumes and expected contaminant loadings to land-use types. Similar land-use types may, however, vary in the amount of impervious areas connected to the stormwater drainage system because of inconsistencies in local development practices. Also, local drainage practices vary. The type of runoff control practices that predominate in one catchment may not be consistent with other similar land use catchments. For instance, catchments with curb and gutter drainage controls will convey larger amounts of runoff of lesser quality than catchments with grassed swales.

Although there exists a wide variety of drainage practices and development practices within catchments of similar land use, land use can still be an important indicator of general stormwater quality. Most development data are reported by land-use category. Land use can, therefore, be a significant predictive tool for urban hydrologists. Table 5-2 presents typical urban pollutant yields based on land use categories.

Total annual loading data are also affected by annual precipitation and intensity characteristics. Since most references normalize their data by area and land use, but not by precipitation, there is a wide variation of concentrations and loads reported in the literature. Extrapolation of data from one study area to another should only be done by individuals who are familiar with the water quality characteristics of urban runoff.

5.2 Water Quality Aspects of Stormwater

The potential for contamination of surface waters through stormwater runoff is, of course, very high given the levels of contaminants sometimes associated with the runoff. Uncontrolled runoff means that the majority of the contaminants will be discharged directly to a receiving stream. The levels of the various contaminants will be affected by a number of factors, however, including the size and intensity of the runoff event, the time interval between events, and the time of year.

As indicated in Tables 5-1 and 5-2, stormwater from street runoff and other impervious surfaces combined with runoff from pervious ground areas such as lawns, parks, and agricultural land can contain a number of different contaminants in relatively high concentrations. These contaminants can have significant impact on the quality of receiving streams.

Table 5-1
Urban Runoff Characteristics

Parameter	Seattle[2] Washington	Lake Ellyn[2] Michigan	Event Mean Conc[3] USA	Peak Conc[4] USA	City of Edmonton[5]			
					Summer		Spring	
					Groat Road	30th Ave.	Groat Road	30th Ave.
Conductivity, mohm/cm	12.9		-		21.8			-
Turbidity, NTU	7.0							
DO, mg/L	9.0							
BOD, mg/L	30.4	18.0	12 - 19		20.9	14.6	48.6	29.7
COD, mg/L	99.0		82 - 178				511.9	204.4
Chloride, mg/L	11.6	34.7			8.4		135.5	70.3
Sulphate, mg/L	20.0							
Nitrogen, mg/L			1.90 - 4.18					
Organic	1.71							
Ammonia	0.35	0.18			0.80	0.58	1.9	1.6
Nitrite	0.13							
Nitrate	0.74				0.57	1.04	1.0	1.5
Phosphorus, mg/L			0.42 - 0.88		0.65	0.55	2.8	1.4
Hydrolyzable	0.36	0.08						
Ortho	0.11		0.15 - 0.28		0.07		0.8	0.7
Lead, mg/L	0.360	0.224	0.182 - 0.443	0.460	0.024	0.012	0.21	0.14
Iron, mg/L	1.99				7.1	2.56	73.4	44.7
Mercury, mg/L	0.17							
Arsenic, mg/L	-			0.051				
Copper, mg/L	-	0.041	0.043 - 0.118	0.100	0.019	0.008	0.17	0.11
Cadmium, mg/L	0.015			0.014	0.0006	<0.0005	0.005	0.005
Zinc, mg/L	0.120	0.171	0.202 - 0.633	2.400	0.101	0.075	0.961	0.631
Phenols, mg/L				0.115	0.01	0.01	0.06	0.05
a-BHC, mg/L				0.1				
Solids, mg/L								
Settleable	121							
Suspended	160	196	180-548		178.0	210.6	1056.7	297.7
TDS, mg/L	144							
Coliforms, org./100 ml								
Total	26,000				3,010,000		714,000	700,000
Fecal	1,200				121,667		51,000	23,000

Notes:
[1]From Kibler (1982)
[2]From Hey and Schaefer (1984)
[3]From NURP
[4]From Cole et al (1984)
[5]From City of Edmonton (1995)

Table 5-2
Sheetflow Quality Summary for Source Areas (mean concentration and reference)

Pollutant and Land Use	Roofs	Paved Parking	Paved Storage	Unpaved Parking/Storage	Paved Driveways	Unpaved Driveways	Dirt Walks	Paved Sidewalks	Streets
Total Residue (mg/l)									
Residential:	58 (5)	1790 (5)	73 (5)		510 (5)		1240 (5)	49 (5)	325 (5)
	64 (1)								235 (4)
	18 (4)								
Commercial:	95 (1)	340 (2)							325 (4)
	190 (4)	240 (1)							
		102 (7)							
Industrial:	113 (5)	490 (5)	270 (5)	1250 (5)	506 (5)	5620 (5)		580 (5)	1800 (5)
Particulate Residue (mg/l)									
Residential:	22 (1)	1660 (5)	41 (5)		440 (5)		810 (5)	20 (5)	242 (5)
	13 (5)								
Commercial:		270 (2)							242 (5)
		65 (1)							
		41 (7)							
Industrial:	4 (5)	306 (5)	202 (5)	730 (5)	373 (5)	4670 (5)		434 (5)	1300 (5)
Filterable Residue (mg/l)									
Residential:	42 (1)	130 (5)	32 (5)		70 (5)		430 (5)	29 (5)	83 (5)
	5 (5)								83 (5)

Commercial:		70 (2)							83 (5)
		175 (1)							
		61 (7)							
Industrial:	109 (5)	184 (5)	68 (5)	520 (5)	133 (5)	950 (5)		146 (5)	500 (5)
BOD (mg/l)									
Residential:	3 (4)	22 (4)							13 (4)
Commercial:	7 (4)	11 (1)							
		4 (8)							
Industrial:									
COD (mg/l)									
Residential:	46 (5)	173 (5)	22 (5)		178 (5)			62 (5)	174 (5)
	27 (1)								170 (4)
	20 (4)								
Commercial:	130 (4)	190 (2)							174 (5)
		180 (4)							
		53 (1)							
		57 (8)							
Industrial:	55 (5)	180 (5)	82 (5)	247 (5)	138 (5)	418 (5)		98 (5)	322 (5)

Total Phosphorous (mg/l)									
Residential:	0.03 (5)				0.36 (5)		0.20 (5)	0.80 (5)	0.62 (5)
	0.05 (1)								0.31 (4)
	0.1 (4)								
Commercial:	0.03 (4)	0.16 (1)							0.62 (5)
	0.07 (4)	0.15 (7)							
		0.73 (5)							
		0.9 (2)							
		0.5 (4)							
Industrial:	<0.06 (5)	2.3 (5)	0.7 (5)	1.0 (5)	0.9 (5)	3.0 (5)		0.82 (5)	1.6 (5)
Total Phosphate (mg/l)									
Residential:	<0.04 (5)				<0.2 (5)		0.66 (5)	0.64 (5)	0.07 (5)
	0.08 (4)								0.12 (4)
Commercial:	0.02 (4)	0.03 (5)	<0.02 (5)						0.07 (5)
		0.3 (2)							
		0.5 (4)							
		0.04 (7)							
		0.22 (8)							
Industrial:	<0.02 (5)	0.6 (5)	0.06 (5)	0.13 (5)	<0.02 (5)	0.10 (5)		0.03 (5)	0.15 (5)
TKN (mg/l)									
Residential:	1.1 (5)				3.1 (5)		1.3 (5)	1.1 (5)	2.4 (5)
	0.71 (4)								2.4 (4)

Commercial:	4.4 (4)	3.8 (5)							2.4 (5)
		4.1 (2)							
		1.5 (4)							
		1.0 (1)							
		0.8 (8)							
Industrial:	1.7 (5)	2.9 (5)	3.5 (5)	2.7 (5)	5.7 (5)	7.5 (5)		4.7 (5)	5.7 (5)
Ammonia (mg/l)									
Residential:	0.1 (5)	0.1 (5)	0.3 (5)		<0.1 (5)		0.5 (5)	0.3 (5)	<0.1 (5)
	0.9 (1)								0.42 (4)
	0.5 (4)								
Commercial:	1.1 (4)	1.4 (2)							<0.1 (5)
		0.35 (4)							
		0.38 (1)							
Industrial:	0.4 (5)	0.3 (5)	0.3 (5)	<0.1 (5)	<0.1 (5)	<0.1 (5)		<0.1 (5)	<0.1 (5)
Phenols (mg/l)									
Residential:	2.4 (5)	12.2 (5)	30.0 (5)		9.7 (5)		<0.4 (5)	8.6 (5)	6.2 (5)
Industrial:	1.2 (5)	9.4 (5)	2.6 (5)	8.7 (5)	7.0 (5)	7.4 (5)		8.7 (5)	24 (7)
Aluminum (ug/l)									
Residential:	0.4 (5)	3.2 (5)	0.38 (5)		5.3 (5)		<0.03 (5)	0.5 (5)	1.5 (5)
Industrial:	<0.2 (5)	3.5 (5)	3.1 (5)	9.2 (5)	3.4 (5)	41 (5)		1.2 (5)	14 (5)
Cadmium (ug/l)									
Residential:	<4 (5)	2 (5)	<5 (5)		5 (5)		<1 (5)	<4 (5)	<5 (5)
	0.6 (1)								

Commercial:		5.1 (7)							,5 (5)
		0.6 (8)							
Industrial:	<4 (5)	<4 (5)	<4 (5)	<4 (5)	<4 (5)	<4 (5)		<4 (5)	<4 (5)
Chromium (ug/l)									
Residential:	<60 (5)	20 (5)	<10 (5)		<60 (5)		<10 (5)	<60 (5)	<60 (5)
	<5 (4)	71 (4)							49 (4)
Commercial:	<5 (4)	19 (7)							<60 (5)
		12 (8)							
Industrial:	<60 (5)	<60 (5)	<60 (5)	<60 (5)	<60 (5)	70 (5)		<60 (5)	<60 (5)
Copper (ug/l)									
Residential:	10 (5)	100 (5)	20 (5)		210 (5)		20 (5)	20 (5)	40 (5)
	<5 (4)								30 (5)
Commercial:	110 (4)	40 (2)							40 (5)
		46 (4)							
		110 (7)							
Industrial:	<20 (5)	480 (5)	260 (5)	120 (5)	40 (5)	140 (5)		30 (5)	220 (5)
Lead (ug/l)									
Residential:	<40 (5)	250 (5)	760 (5)		1400 (5)		30 (5)	80 (5)	180 (5)
	30 (3)								670 (4)
	48 (1)								
	17 (4)								
Commercial:	19 (4)	200 (2)							180 (5)
	30 (1)	350 (3)							
		1090 (4)							

		146 (1)							
		255 (7)							
		54 (8)							
Industrial:	<40 (5)	230 (5)	280 (5)	210 (5)	260 (5)	340 (5)		<40 (5)	560 (5)
Zinc (ug/l)									
Residential:	320 (5)	520 (5)	390 (5)		1000 (5)		40 (5)	60 (5)	180 (5)
	670 (1)								140 (4)
	180 (4)								
Commercial:	310 (1)	300 (5)							180 (5)
	80 (4)	230 (4)							
		133 (1)							
		490 (7)							
Industrial:	70 (5)	640 (5)	310 (5)	410 (5)	310 (5)	690 (5)		60 (5)	910 (5)

References:
[1] Bannerman et. al. 1983 (Milwaukee) (NURP)
[2] Denver Regional Council of Governments 1983 (NURP)
[3] Pitt 1983 (Ottawa)
[4] Pitt & Boxeman 1982 (San Jose)
[5] Pitt & McLean 1986 (Toronto)
[7] STORET Site #590866-2954309 (Shop-Save-Durham, NH) (NURP)
[8] STORET Site #596296-2954843 (Huntigton-Long Island, NY) (NURP)

Stormwater flows in developed areas may also alter natural hydrologic conditions that exist within receiving water bodies. These alterations can negatively affect stream quality.

Stormwater control practices can significantly affect the quality of surface water. It is important therefore to consider the quality of the stormwater in association with the preferred method of control to make certain that no deterioration in either surface water or groundwater results.

5.2.1 Surface Waters

5.2.1.1 Alteration of Hydrologic Conditions

The overall hydrologic impact of discharging uncontrolled stormwater to a receiving stream is normally to increase peak flows and decrease low flows. Stormwater discharged directly to a receiving stream increases the peak flow volumes normally experienced, which may also negatively affect erosion patterns and change channel geometries. Low flow conditions experienced in developed areas limits available aquatic habitat and may concentrate contaminants through increased deposition.

5.2.1.2 Sediment Loading

Sediment loading is perhaps the most predominant pollutant category associated with stormwater runoff. Sediment from both pervious and impervious areas as well as from new development or construction sites may negatively affect receiving water quality.

Sediment transported to a stream may affect water clarity thus reducing light penetration and altering the rates of photosynthesis in aquatic plants. Sediment deposited in a receiving water can limit the availability of viable spawning and rearing habitats for some fish species. A number of nutrients and other contaminants associated with sediment may be transported to a receiving water, causing significant deterioration of water quality.

Ultimately, sediment deposited in a receiving stream may alter the conveyance and storage capacities of the stream, resulting in channel modifications through increased rates of erosion and flooding.

5.2.1.3 Nutrient Enrichment

Urban stormwater runoff may contain concentrations of nutrients such as phosphates and nitrates above those normally expected in undeveloped areas. High nutrient loadings to receiving waters may increase eutrophication rates. The excessive growth of aquatic plants found under eutrophic conditions may also limit the dissolved oxygen content in the water body.

5.2.1.4 Toxicity

Many contaminants found in urban stormwater can be directly toxic to aquatic organisms. Some forms of heavy metals such as copper, zinc and cadmium are toxic to aquatic organisms if present at concentrations above water quality objectives. Many types of pesticides and herbicides may also be found in stormwater runoff and can be toxic to aquatic organisms.

5.2.1.5 Microorganisms

Bacteria are often found in urban stormwater at levels above water quality objectives. The elevated levels may be due to cross-connected sanitary systems or combined sanitary systems, or from animal/bird waste. Elevated levels of bacteria such as *Fecal coliform* and *E-coli* can result in recreational impairment of a water body.

5.3.1.6 Salt

In urban areas where salt is used during the winter months to de-ice roadways, the impact of runoff during periods of snowmelt can be quite significant. Salt concentrations are generally not high enough to be directly toxic to fish and invertebrate species but can negatively affect the growth of aquatic plant species.

5.2.1.7 `Water Temperature

Increased direct runoff of stormwater to a receiving water can increase stream temperatures to a level that limits the habitat viability for aquatic organisms that are endemic to particular stream reaches. Fish species and aquatic invertebrates have temperature preferences that may be exceeded during periods of stormwater runoff.

5.2.2 Groundwater

Stormwater control practices often depend to some degree, and in some cases to a large degree, on the infiltration of stormwater. There is, therefore, significant potential for groundwater contamination through the implementation of stormwater controls that rely to some degree on infiltration. The following is a summary of research carried out on the potential for groundwater contamination through infiltration.

5.2.2.1 `Microorganisms

Most bacterial organisms are normally retained in the upper soil layers. Viruses may enter groundwater through infiltration much more readily than bacteria. Viruses have been found at concentrations above background levels in groundwater beneath infiltration basins where the aquifer is close to the surface.

5.3.2.2 Metals

Most metals including aluminum, arsenic, cadmium, chromium, copper, iron, lead, mercury, and nickel are normally associated with particulate fractions and are therefore mostly removed by filtration or sedimentation. The effectiveness of stormwater control alternatives in limiting the infiltration of metals is dependent on, among other factors, soil characteristics of grain size, void ratio and homogeneity.

5.2.2.2 Nutrients

Of the two most common nutrients, nitrogen and phosphorus, nitrates most readily and rapidly infiltrate into groundwater. Nitrates are one of the most frequently encountered contaminants in groundwater.

5.2.2.4 Organics

Groundwater contamination from organics, like most other pollutants, occurs more readily in areas with sandy soils. Sources of organics include urban and rural pesticide/herbicide use and industrial facilities.

5.2.2.5 Salts

Salts applied to road surfaces collect in snowmelt and travel through soil layers to groundwater with very little attenuation.

5.3 The Need for Water Quality Analysis

Rigorous analysis of the quality of urban runoff requires the collection and assessment of a great deal of data. It is usually feasible to conduct such a thorough analysis for only those situations where stormwater runoff has been recognized as having the potential to cause receiving water impairment in critical areas. Simple and approximate alternatives have been developed to address most common stormwater runoff situations, recognizing that the results will be subject to some degree of uncertainty.

The need for water quality analysis should be considered in two broad categories. First, if a municipal water supply, recreational area, or particularly sensitive biological resource is likely to be affected, there is need for a fairly comprehensive analysis. Similarly, if there is potential to significantly aggravate an existing water quality problem, such analysis may be justified. Detailed study would necessarily include extensive data collection taking into account event, seasonal, and annual variation in a broad range of contaminant concentrations. Continuous computer simulations that include multiple events are required to accurately estimate the magnitude and frequency of loadings and their ultimate fate. The computer simulations also allow evaluation of the effects of various control or treatment measures, such as sewer diversion or detention ponds.

The second category comprises stormwater discharges which do not, when assessed by themselves, represent a significant receiving stream impact but whose cumulative effects may be of concern. The scale of most land development projects in Alberta is too small to cause substantial water quality impacts by themselves or to justify the cost of extensive water quality studies. The potential for serious problems to arise due to the cumulative impact of multiple development should be identified at the river basin planning or watershed drainage plan levels. Consideration should be given to both the costs of maintaining water quality and the risks associated with making inadequate provision for stormwater quality control, recognizing the high cost of future remedial measures.

Most potential water quality problems fall into the second category where the uses of the receiving water are not vital and there is no evidence of immediate serious water quality degradation. In such instances, the use of stormwater ponds for water quality control in new development should be encouraged. If in these instances, ponds will be required for flood or downstream erosion control, their benefits in improving water quality should be considered as a design criteria. The additional water quality criteria as design considerations will, in most instances, involve only a small additional expense.

In evaluating the significance of stormwater pollution, both the loadings and the concentrations of contaminants must be considered. Loadings are usually expressed in kilograms per year. Since stormwater pollution events are random in nature (depending on runoff intensity, inter-event times, and climatic variables), consideration must be given to the magnitude and distribution of loads over a number of years. The characteristics of a receiving water must be evaluated in terms of its ability to assimilate and dilute the loads imposed upon it. Concentrations are important for this as the assimilation must be evaluated in terms of receiving-stream objectives. These objectives are usually expressed as concentrations for those contaminants of concern.

5.3.1 Associated Data Collection Requirements

In situations where the water quality impacts on the receiving watercourse are of particular concern, a water quality sampling and simulation study should be conducted. Such a study requires several seasons for the collection of field data and a significant budget for the analysis of data collected from previous work that may span several years. Several seasons of site-specific field-data collection are necessary because of the extreme range of environmental conditions that can be encountered. In Alberta, rainfall events are quite variable at different times of the year and winter/spring runoff conditions can vary from year to year. Short periods of data collection do not yield sufficient data for model calibration and verification. The filed data collection activities provide data points which can be extrapolated to represent design conditions.

5.3.2 Rainfall Data

Site-specific rainfall data should be collected from a number of locations within the catchment area being studied. The rainfall data are collected for specific storm events and used to calibrate runoff simulation models. Historical rainfall data should be collected from existing monitoring locations to determine the long-term trends in frequency, duration, and intensity of rainfall events. Rainfall data collected over a minimum period of 10 to 20 years are required for long-term evaluation of trends.

5.3.3 Flow Data

If a stormwater collection system already exists (existing development area), flow data from strategic discharge points should be collected in association with the rainfall data during specific storm events. These data should be collected over several seasons for representative storm events and used to calibrate/verify the simulation model. This allows the models to be used with a reasonable degree of confidence for continuous runoff simulations.

5.3.4 Runoff Quality

Contaminant concentration data may also be collected from strategic locations within the catchment area being studied. The range of contaminants to be sampled depends on the particular land use in the catchment area of concern and the nature of the receiving environment. Generally, water quality sampling programs for urban development include the following parameters:

- Total suspended solids (TSS),
- Total dissolved solids (TDS),
- Biochemical oxygen demand (BOD),
- Total phosphorus (TP),
- Total nitrogen (TN),
- Nitrate (NO_3),
- Chloride (Cl),
- Lead (Pb),
- Zinc (Zn),
- Total coliforms,
- E-coli, and
- Faecal coliforms

Note that sampling programs and parameters for industrial discharges are not covered by these guidelines.

5.3.5 Sampling Protocols

In new development areas the sampling program should be conducted on catchments having a similar land use adjacent to the area of planned development. The program must capture data for a full range of runoff events (large and small) during the study period. Complete

runoff events must be sampled. Missing data for contaminant washoff during the earlier part of the storms will prevent the proper material balances from being calculated. The high concentration of many contaminants in the early part of storms, referred to as the "first flush", is a common phenomenon. It is caused by the rapid mobilization of contaminants attached to fine sediments on impervious surfaces and by the flushing of catchbasins and manholes by the first runoff from a storm.

Obtaining data from the early portion of a runoff event poses difficult logistical problems because of the need for rapid mobilization of sampling crews in response to incoming storms. A substantial budget is required to keep crews in a state of readiness, and many "dry" runs should be anticipated as crews arrive before forecasted rainfalls that do not materialize. The need for night and weekend data collection is a further logistical problem. Automatic sampling equipment is a potential solution, but requires extensive setup, calibration, and monitoring. Although new equipment is becoming more reliable, it is still prone to failure and requires frequent checking and maintenance.

5.4 Modelling

Several years of data collection are often inadequate to characterize contaminant loadings over a broad range of environmental conditions. Data collection, analysis, and monitoring programs can also be extremely costly. The data collected from a less extensive but more intensive sampling program can, however, be used to calibrate computer models which simulate the loading of contaminants over a more lengthy period. There is a broad range of available computer models for the assessment of stormwater runoff loadings. Five commonly used computer software packages that calculate continuous simulations of hydrologic conditions and water quality are the U.S. Army Corps of Engineers' STORM model, the U.S. EPA's SWMM 4.3 model, the U.S. EPA's HSPF model, U.S. EPA's WASP, and the U.S. EPA's QUAL2E-U model.

Each model is capable of the long-term continuous simulation necessary for the analysis of contaminant loads. The STORM model and SWMM model are used primarily for the assessment of runoff conditions in a given stormwater drainage basin. WASP and QUAL2E are used primarily for the assessment of receiving-stream impacts resulting from stormwater runoff. HSPF can address both runoff quality and instream impacts. The selection of an appropriate model and the complexity of the analysis is determined by the nature of the problem and the type of drainage system under study, the type of analysis required, the objectives or required outcome, and the data and budget available for the work.

5.4.1 Modelling Considerations

Once a model has been selected, data and model limitations must be considered. Water quality models are comparatively complex, so experienced personnel are required for their application. Since quality models are significantly less accurate than quantity models, expectations for the final study must reflect these limitations to avoid high modelling costs that do not yield the anticipated results. The following points should be kept in mind when designing a water quality modelling study:

- Initial modelling efforts should be kept as simple as possible. Approximate results may be adequate for decision-making, particularly if the costs of more sophisticated modelling are prohibitive. Where detailed modelling is planned from the outset, coarse screening models should still be used to test whether the detailed simulation will yield useful results.

- Lumped catchment modelling is often advisable, as the quality of the available runoff and contaminant loading data normally limits the accuracy of the modelling. Detailed discretization usually will not yield further information.

- Continuous simulation, at least during the screening process, is essential. The rate of contaminant buildup and the antecedent moisture conditions have a major effect on loadings. Analysis of contaminant accumulation requires the use of continuous simulation.

- Selected design storms may be of use in testing treatment facility design and the impact on receiving waters once continuous simulations have been completed. The design storms can be established through frequency analysis of the output from the continuous simulation modelling.

- Calibration procedures should concentrate first on establishing a good match of runoff volumes and peaks, often by examining a subset of single event simulations. Calibration of contaminant buildup and washoff can then be undertaken in a continuous mode.

- It is generally advisable to limit contaminant simulation to a few parameters such as TSS, TDS, BOD, TN, and coliforms. Other contaminants can often be associated with the loading function for one of these parameters.

The above comments relate to urban runoff modelling for estimating contaminant loads. Studies that include modelling receiving-stream impacts and contaminant transport and reaction processes are considerably more complex and require careful planning and execution.

5.4.2 Urban Runoff Water Quality Models

5.4.2.1 STORM

The STORM model normally has the advantages of lesser data requirements and simulation costs than the other models. Used as a water quality model, it is used primarily to generate hourly contaminant loadings to the receiving water. The two types of output generated by the model include information on the quantity of runoff and the quality of runoff (pollutographs) over the simulation period. STORM is limited in its assessment of water quality parameters to a select number and type of pollutants.

The Storm model has been adapted to run continuous simulations using hourly rainfall records over a 1-year rainfall record. The STORM model provides estimates of the mean stormwater runoff frequencies and volumes over the simulation period. This analysis allows the assessment of the mean performance of various stormwater control alternatives. This analysis can be used to size storage and treatment elements for runoff quantity and quality control. The model considers seven elements in its control analysis:

- Rainfall/snowmelt
- Runoff
- Dry weather flow
- Pollutant accumulation and runoff
- Land surface erosion
- Treatment rates
- Detention storage

A typical STORM model assessment may include altering the treatment rate and land use, and taking note of which changes have occurred in the system response. Hydrologic applications of STORM have been described in Section 4.

5.4.2.2 SWMM 4.3

The SWMM 4.3 program is more complex in its characterization of washoff processes and stormwater runoff systems. SWMM is adynamic model capable of simulating varying flows and various points of discharge. The model can simulate the loading of a wider range of pollutants than the STORM model. Also, SWMM 4.3 can more readily simulate storage treatment facilities such as retention ponds and receiving-water impacts and can be used to determine both the impacts of stormwater runoff and the effectiveness of treatment facilities. The SWMM 4.3 model is comprised of a number of "Blocks". The RUNOFF block generates surface and subsurface runoff based on rainfall and/or snowmelt hyetographs antecedent conditions, land use, and topography. The SWMM 4.3 model is capable of very complex routing using the RUNOFF and TRANSPORT blocks which route pollutant flows through the drainage system. The STORAGE AND TREATMENT block simulates the implementation of various stormwater control alternatives. Output from the model includes hydrographs and pollutographs. Hydrologic applications of SWMM 4.3 have been described in Section 4.

5.4.2.3 QUALHYMO

QUALHYMO is a planning level continuous water quality and quantity simulation program developed in 1983 at University of Ottawa for the analysis of stormwater and pollutant runoff. The structure of QUALHYMO is based on the HYMO and OTTHYMO programs discussed in Section 4.6.5 It consists of a series of subroutines for simulating rainfall and runoff from a rural or undeveloped watershed to an urbanized watershed. It can carry out continuous simulation of rainfall and runoff, and routing through detention pond and natural channels for water quality control and assessment. The constituents that can be simulated

are stormwater runoff consisting of pollutants with first order decay and sediments of varying size fractions. The components which the program can simulate are catchments, detention ponds, river reaches, and flow diversions. Some of the typical commands that the program uses are: GENERATE to generate runoff and pollutant loads, POND to route flows and pollutants through a detention storage pond, POLLUTANT RATES to establish pollutant source rates and appropriate parameters, CALIBRATE to compare two pollutant or flow series, and EXCEEDANCE CURVES to calculate the number and duration of flow and concentration exceedances.

5.4.3 Receiving Stream Water Quality Models

5.4.3.1 HSPF

HSPF is the most sophisticated and powerful of the models described here, but has comparably high data requirements and computing costs. HSPF can simulate loadings, effectiveness of treatment, and receiving-water response. Since HSPF can also simulate point and rural non-point contaminants, it can be used in the analysis of complex watersheds involving a wide variety of contaminant types and sources. The continuous rainfall input to HSPF is divided into interception losses, infiltration, and runoff from impervious surfaces to compute a continuous hydrograph of flow at the catchment outlet. Total stream flow is calculated as a combination of surface runoff, interflow, and groundwater flow. HSPF allows an integrated analysis of these runoff characteristics with receiving-water impacts. The model can simulate an extremely wide range of contaminants as well as receiving-water reactions including hydrolysis, oxidation, biodegradation, volatilization, and sorption.

5.4.3.2 QUAL2E-U

QUAL2E-U is a comprehensive and versatile stream water quality model. QUAL2E is capable of simulating a wide range of pollutant types through a mass transport analysis of branching stream systems. The water quality constituents that QUAL2E-U can simulate include:

- Dissolved oxygen
- Biochemical oxygen demand
- Temperature
- Chlorophyll a
- Ammonia
- Nitrite
- Nitrate
- Phosphate
- Coliforms
- One nonconservative constituent
- Three conservative constituents

QUAL2E-U provides a hydrologic balance, a heat balance and a materials (pollutant) balance at each defined reach. QUAL2E-U considers both advective and dispersion transport processes in the analysis of contaminant fate and transport. QUAL2E-U however, does not simulate flow conditions that vary with time.

5.4.3.3 WASP5

The Water Quality Analysis Simulation Program-5 (WASP5) is an enhancement of the original WASP program developed in the 1980's. The WASP5 system is a program that can model aquatic systems, including the water column and the underlying benthos. The program can help users to interpret and predict water quality responses to natural and man-made pollution for pollution management decisions.
The WASP5 system comprises two stand-alone computer programs, DYNHYD5 and WASP5, that can be run in conjunction or separately. The hydrodynamics program, DYNHYD5, simulates the movement of water while the water quality program WASP5, simulates the movement and interaction of pollutants within the water.

WASP5 is supplied with two kinetic sub-models, EUTRO5 and TOXI5, to simulate two of the major classes of water quality problems: conventional pollution (involving dissolved oxygen, biochemical oxygen demand, nutrients and eutrophication) and toxic pollution (involving organic chemicals, metals, and sediment)

WASP5 analyzes a variety of water quality problems in water bodies such as ponds, streams, lakes, reservoirs, rivers, estuaries, and coastal waters.

6.0 Stormwater Best Management Practices (BMPs)

6.1 Introduction

6.1.1 General

Stormwater Best Management Practices (BMPs) are methods of managing stormwater drainage for adequate conveyance and flood control that are economically acceptable to the community. BMPs are stormwater management methods that retain as much of the "natural" runoff characteristics and infiltration components of the undeveloped system as possible and reduce or prevent water quality degradation.

The selection and design of stormwater BMPs must incorporate water quantity and water quality concerns. Many common stormwater management practices are limited in terms of the environmental benefits they provide. Common practices such as armoured channels and direct discharge outfalls are good conveyance practices but may result in substantially more detriment to the environment than environmental benefit. Most designers of stormwater management facilities now recognize that stormwater quality and the impact of stormwater management facilities on the environment are important factors to consider in their selection of management practices.

BMPs that address source controls such as street sweeping, catchbasin cleaning and anti-litter regulations should be a component of specific drainage plans. Source controls can have a significant effect on the total contaminant load discharged to a receiving water body. The optimum frequency of street sweeping and catchbasin cleaning depends on the nature of the development. Anti-litter regulations are normally a component of a municipal bylaw that addresses all forms of development. Source controls alone do not reduce the total contaminant loads to acceptable levels in most development areas. It is important to consider further treatment and runoff controls in the selection of BMPs.

It is recognized that most annual urban runoff quantities are the result of more common smaller storms. Large storms are, however, the focus of most drainage design because they represent the most significant conveyance problems. In terms of water quality, the more frequent small storms also represent the largest pollutant load to receiving waters. Large storms contribute significant contaminant loadings but usually over a limited time period.

The application of BMPs to stormwater management requires the consideration of a new comprehensive set of criteria. These criteria include all aspects of traditional conveyance practices and incorporate additional environmental criteria that are selected to preserve hydrologic conditions and water quality. This section reviews the types of BMPs that are in common practice and discusses their performance as well as design and selection considerations.

While the BMPs presented in these guidelines relate primarily to stormwater control in the final development, it is just as important that measures be taken to control stormwater during the construction of the development. Temporary sediment and erosion controls should be

installed to protect adjacent areas. Sediment and erosion controls are particularly important during construction. Temporary perimeter drainage swales directed to a temporary detention pond, filter cloth and check dams, infiltration catchbasins, timed staging of excavation, protection and maintenance of vegetation covers, are some of the techniques applied.

As described previously in Section 3, good planning and design integrates the design of a site and the design of the stormwater management facilities into one process. Similarly, the integration of (BMPS) into the planning and design process of the drainage system is essential if an effective stormwater management plan is to happen. The BMPs described in this section should be applied when designing the drainage system.

Although BMPs are presented as individual elements, they should be used either as stand alone facilities or in combination when designing the overall drainage system for the particular site. Site specific conditions and characteristics will govern the stormwater management solutions given in these guidelines. It is be up to the designer's experience and judgement, and the requirements of the local regulatory agencies, to design an appropriate stormwater management plan.

Stormwater BMPs that may be considered for stormwater quantity and quality controls are discussed in the following order:

- Source control BMPs
- Lot-level BMPs
- Conveyance system BMPs
- End-of-pipe BMPs

6.1.2 Design Criteria for Stormwater Quality Control

The "first flush" runoff from a storm is commonly thought to be the most contaminated. Also, a study in the U.S. of cities with widely varying climatic conditions revealed that most of the runoff in urban areas is generated by small storms, that is, storms smaller than the 4-month storm, and generally produce less than 12 mm of runoff. The study indicates that, in most cases, less than 12 mm of storage is required to capture 90 percent of the runoff and that 25 mm of storage is required to capture over 95 percent of the runoff (by volume, on an average annual basis). Studies carried out in Ontario also indicate that storing the first 25 mm of a storm would result in a capture rate of 95 percent of the annual precipitation. In Ontario, they found that storing the 25 mm for 24 hours also serves to mitigate erosion concerns. In Edmonton they found that in areas with 60 percent directly connected impervious area, about 80 percent of the average annual rainfall is captured by storage sized to capture the first 10 mm of runoff.

In the absence of detailed studies in Alberta, it is considered that providing 25 mm of storage for the contributing area is appropriate for Alberta for stormwater quality control using detention devices such as dry ponds, wet ponds, and constructed wetlands. A detention time of 24 hours should also be used for detention facilities since it is well established that for a detention basin to be effective as a quality control device, the detention time must be 24

hours or greater. Based on the same studies, using the runoff from a 12-mm storm event over the contributing area is considered appropriate for infiltration BMP's.

6.2 Source Control BMPs

Removal of stormwater contaminants at their source may, in some instances, be a practical solution to the mitigation of pollutant impacts. There are three main pollutant removal activities that are normally practised by a municipality for source control including street sweeping, catchbasin cleaning, and animal litter removal.

Street sweeping removes a portion of the pollutants deposited on road or parking lot surfaces and thereby reduces pollutant washoff from stormwater to combined sewers and storm sewers, and subsequently to receiving waters. The effectiveness of street sweeping in reducing pollutant loadings is dependent on a variety of factors, including time of year, frequency of service, length of time between rainfall events, type of sweeping equipment (vacuum, wet, dry etc.) and the type of road surface. Street sweeping is most effective in the early spring to remove the accumulated winter street pollutant loads.

Street sweeping must be carried out very frequently (daily) before it results in a significant reduction in pollutant loadings. Typical street sweeping programs (once or twice per month) remove less than 5 percent of the pollutant loadings. These findings were also confirmed in the TAWMS Studies carried out by the Ontario Ministry of Environment and Energy (MOEE).

Street sweeping is not effective for the control of faecal bacteria. Studies carried out by MOEE for the Rideau River indicated that a maximum of 10 percent of seasonal (rainfall-runoff) loads could be controlled by means of sweeping at an optimum frequency of twice per week. Sweeping has, however, been shown to be more effective in controlling street solids and associated pollutants.

Catchbasin cleaning is the cleaning of accumulated sediments and debris in catchbasin sumps, to reduce the amount of pollutants re-suspended from the sump by stormwater and subsequently discharged to receiving waters. Catchbasin cleaning has not been found to be effective in controlling loadings of faecal bacteria in stormwater runoff. Catchbasins can, however, be effective in trapping larger runoff particulates and associated pollutants. Studies in the Boston area carried out by the U.S. EPA have shown that most cost-effective cleaning frequency is semi-annually.

Municipal bylaws passed to prohibit littering and control disposal of animal wastes may remove a portion of the pollutants deposited on the ground surface and thereby reduce pollutant wash off from stormwater to combined sewers and storm sewers.

The most significant pollutant source that can be addressed by this alternative is dog faeces. Based on previous studies carried out by MOEE for the Rideau River Stormwater Management Plan, dog faeces can contribute significant amounts of faecal coliform bacteria to receiving waters. Furthermore, dog faeces control programs could reduce the amount of

faecal coliform bacteria reaching receiving waters from dog faeces by up to 35 percent. The degree of success of these programs is dependent on public awareness and the degree of enforcement. However, enforcement is difficult due to the large number of dogs and the fact that violators must be caught in the act of breaking the law. This alternative may be more effective on a localized basis if, for example, enforcement is concentrated in park areas that drain via storm sewers to the receiving water.

6.3 Lot-Level BMPs

Stormwater lot-level controls are practices that reduce runoff volumes and/or treat stormwater before it reaches a subdivision/development conveyance system. This type of control can be readily incorporated into the design of future developments. With all development, the applicability of stormwater lot-level controls should be investigated before conveyance and end-of-pipe systems are examined.

Traditional lot-level controls aimed at stormwater quantity management and the reduction of peak runoff rates include:

- Restricting numbers of roof drains to provide rooftop detention to stormwater
- Installing catchbasin restrictors or orifices in the storm sewer to promote parking lot detention
- Oversizing storm sewers and installing orifices in the sewer to create pipe storage
- Installing catchbasin restrictors in rear yard catchbasins to create rear yard storage

The above-noted lot-level measures are primarily designed to reduce runoff peaks. Other stormwater management criteria, such as the preservation of water quality, protection from erosion, and the maintenance of baseflow are not adequately addressed through these techniques. Lot-level controls that help preserve the natural hydrologic regime include:

- Reduced lot grading
- Directing roof leaders to rear yard ponding or soakaway pits
- Sump pumping foundation drains to rear yard ponding areas

6.3.1 Reduced Lot Grading

6.3.1.1 Purpose

The purpose of reducing lot grades is to reduce the volume of runoff from developed lots by increasing the travel time of runoff, and increasing the availability and opportunity for depression storage and infiltration. A significant reduction in lot-level runoff volumes would also affect the other minor stormwater system components and the major system components by reducing the conveyance and treatment requirements.

6.3.1.2 Description

Typical development standards require a minimum lot grade of 2 percent to drain stormwater away from buildings. In flat areas, a reduction to minimum lot grades should be evaluated. In hilly areas, alterations to natural topography should be minimized. To avoid foundation drainage problems, the grading within 2 to 4 m of buildings should be maintained at 2 percent or higher. Areas outside of this envelope should be graded at less than 2 percent.

Reduced lot grading BMPs promote depression storage and natural infiltration and reduces risks associated with flooding and erosion. The maintenance of natural infiltration could have positive impacts on baseflow depending on local evapotranspiration rates.

6.3.1.3 Applicability

Reduced lot grades can be recommended as a lot-level stormwater BMP for any new developments and in regrading or re-landscaping of existing lots in established developments.

6.3.1.4 Effectiveness

Very little information is available in regard to the impact that reductions in lot grades may have on the overall runoff volumes from a developed area. It has been recommended that reductions in lot grading may increase the pervious depression storage by as much as 1.5 mm for a 0.5 percent to 2.0 percent change in grade. Reduction of on-lot runoff will also reduce downstream erosion potential.

6.3.1.5 Water Quantity

Reduced lot gradings limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater also provides recharge to the local groundwater that may, in turn, discharge to local streams thus enhancing baseflows. Reduced lot grading also reduces lot runoff velocities, thereby increasing on-lot sedimentation and on-lot retention of pollutants.

6.3.1.6 Water Quality

Reduced lot gradings limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. The effectiveness of reduced lot grades in limiting contaminant runoff is also dependent on land use.

6.3.1.7 Design Considerations

Design guidelines for on-lot grade reductions are shown in Figure 6-1. Grades within 4 m of structures should be maintained at 2 percent. Grades beyond 4 m of structures should be reduced to 0.5 percent. Consideration should also be given to tilling soils in flatter grade areas to a depth of 30 mm prior to seeding or sodding to reduce soil compaction and increase infiltration potential.

6.3.2 Surface Ponding and Rooftop Storage

6.3.2.1 Purpose

Roof leaders that discharge to surface ponding areas reduce the potential for downstream flooding and erosion and help maintain pre-development end-of-pipe discharge rates. The same benefits can result from the use of rooftop storage, which are likely suitable for commercial, industrial, and institutional buildings.

6.3.2.2 Description

Roof leaders are directed toward rear lot depressions that allow stormwater to infiltrate or evaporate. For rooftop storage roof, drains on flat roofs are raised to allow ponding on the rooftop.

6.3.2.3 Applicability

Surface ponding areas can be recommended as a lot-level stormwater BMP for any new developments and in regrading or re-landscaping of existing lots in established developments. Surface ponding may also be used for parking lots or park areas. Rooftop storage can be recommended for industrial, commercial, or institutional buildings with flat roofs.

6.3.2.4 Effectiveness

Rear lot ponding of stormwater or rooftop storage effectively limits runoff by a volume equal to the amount of impervious depression storage provided.

6.3.2.5 Water Quantity

Rear lot ponding and rooftop storage limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing baseflows.

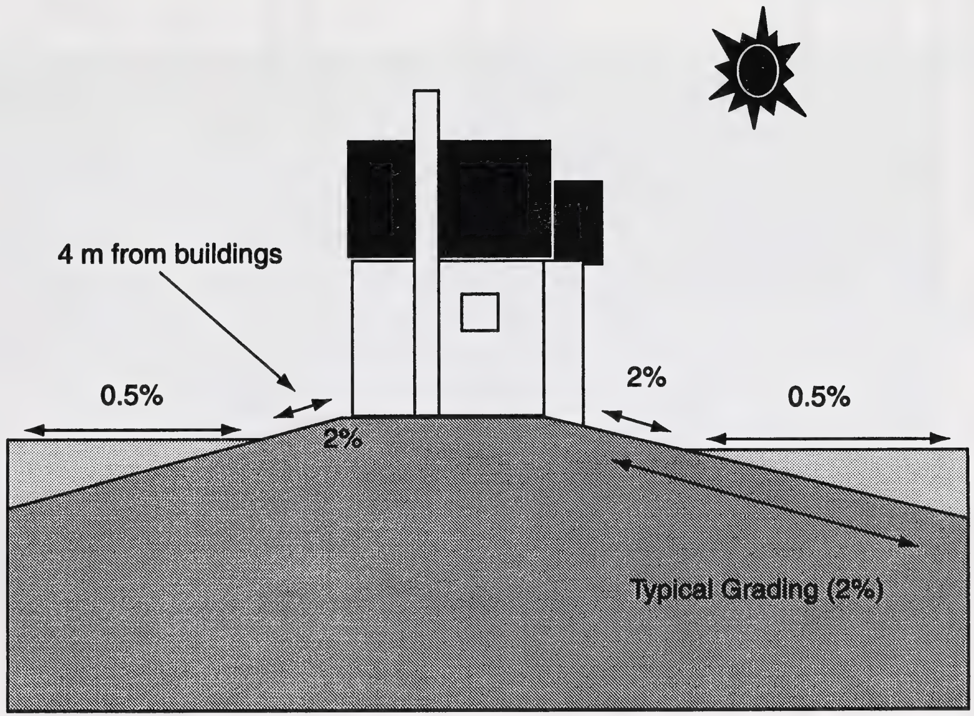


Figure 6.1
Lot Grading Guidelines

6.3.2.6 Water Quality

Rear lot ponding and rooftop storage limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

6.3.2.7 Design Considerations

Design guidelines for rear lot ponds are shown in Figure 6-2. Maximum depths should be maintained at 100 mm. Flow paths should be provided to direct overland flow to the pond. To maintain the pond, catchbasins can be elevated to the required height or grassed swales can be created. More complex designs may incorporate an infiltration trench beneath the ponded area to enhance infiltration. The pond should be sized to accommodate a minimum of 5 mm and a maximum of 20 mm of rainfall covering the roof area. Rooftop ponding can be accomplished by raising roof hoppers to create a maximum ponding depth of 10 mm. Roof supports must be adequate to support the weight of the ponded water.

6.3.3 On-lot Infiltration Systems

6.3.3.1 Purpose

On-lot infiltration systems are used for detention of stormwater from relatively small catchment areas. Infiltration systems may be used in areas without adequate minor system conveyance. They also provide enhancement to water quality and reductions in overland flow.

6.3.3.2 Description

Infiltration systems may be simply designed pits with a filter liner and rock drain material or more complex systems with catchbasin sumps and inspection wells. Stormwater flow from roof drains is directed to the infiltration system.

6.3.3.3 Applicability

Infiltration systems are recommended for relatively small detention volumes. If larger detention volumes are required a series of infiltration basins may be employed. Infiltration basins should not be built under parking lots or other multi-use areas, if the groundwater table is within 0.6 m of the infiltrating surface, if bedrock is located within 1.2 m of the infiltration surface, if the infiltrating surface is located on top of fill material nor if the underlying soils have a fully saturated percolation rate of less than 1.3 mm.

6.3.3.4 Effectiveness

Infiltration systems have a number of advantages over rear yard ponding including increased groundwater recharge and less inconvenience to home owners. Infiltration systems may have increased maintenance requirements over ponds and a more uncertain operating life. On-lot

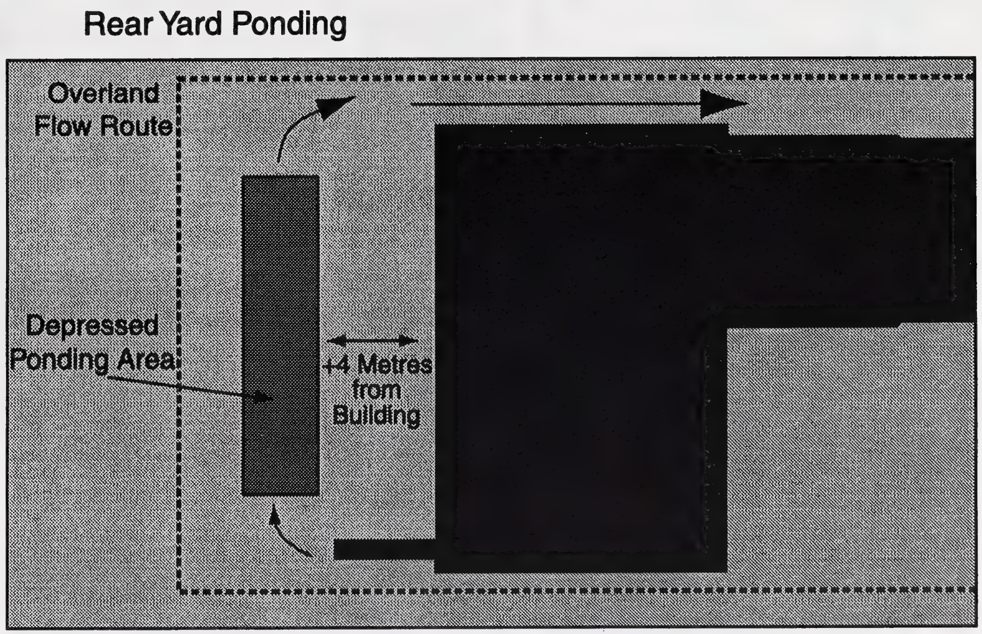


Figure 6.2
Rear Lot Ponding Guidelines

infiltration systems accept only roof runoff and are therefore subjected to minimal levels of suspended solids.

6.3.3.5 Water Quantity

On-lot infiltration systems limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from rear lot ponds also provides recharge to the local groundwater which may in turn discharge to local streams thus enhancing baseflows.

6.3.3.6 Water Quality

On-lot infiltration systems limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants.

6.3.3.7 Design Considerations

Figures 6-3 and 6-4 illustrate two different applications of infiltration systems. The total void volume should be calculated from the storage required for the 2 year design storm which is calculated from the effective porosity of the infiltration fill material. The infiltration surface area required (bottom surface area) to drain the system within 48 hours is calculated from the 24-hour sustained percolation rate. An overland flow path should be provided for overflow volumes during saturated or frozen conditions. A pretreatment filter (Figure 6-3) or sump (Figure 6-4) should be provided to limit solids input into the system. Design void space volumes are calculated from the volume of water required to fill a known volume of drain rock. A suitable quality filter fabric or geotextile must also be incorporated into the design.

In locating infiltration systems, consideration should be given to proximity to septic fields.

6.3.4 Sump Pumping of Foundation Drains

6.3.4.1 Purpose

Many current development standards allow foundation drains to be directly connected to the storm sewer. By pumping foundation drainage to surface or subsurface ponding/soakaway areas, infiltration, flooding, and erosion water management concerns may be reduced.

6.3.4.2 Description

Foundation drainage is sometimes pumped to the storm sewer network, to a suitable infiltration system, or to the surface where it is conveyed to a catchbasin and then to a storm sewer.

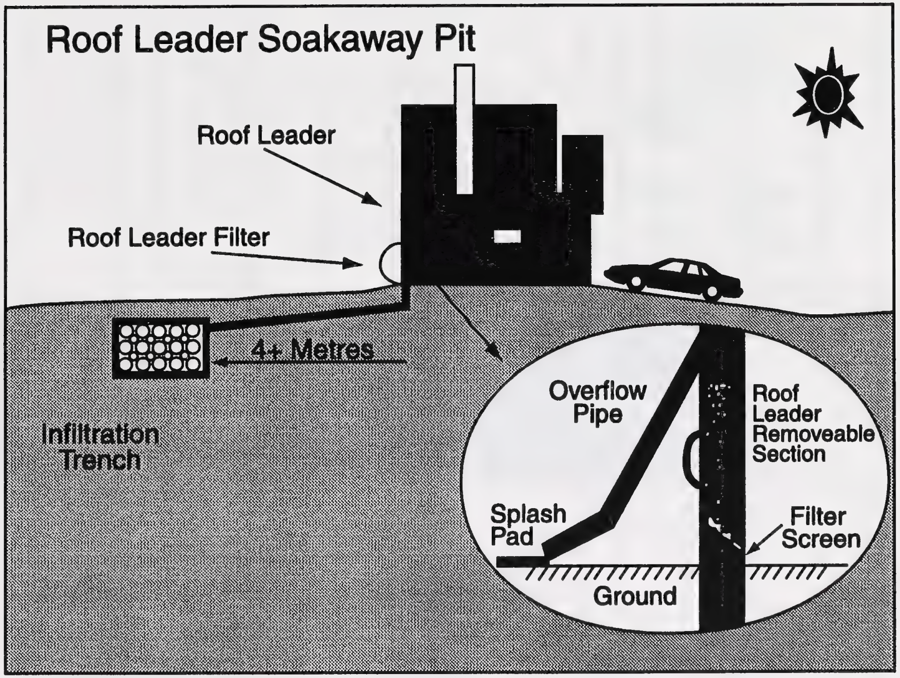


Figure 6.3
Infiltration System with Roof Leader Filter

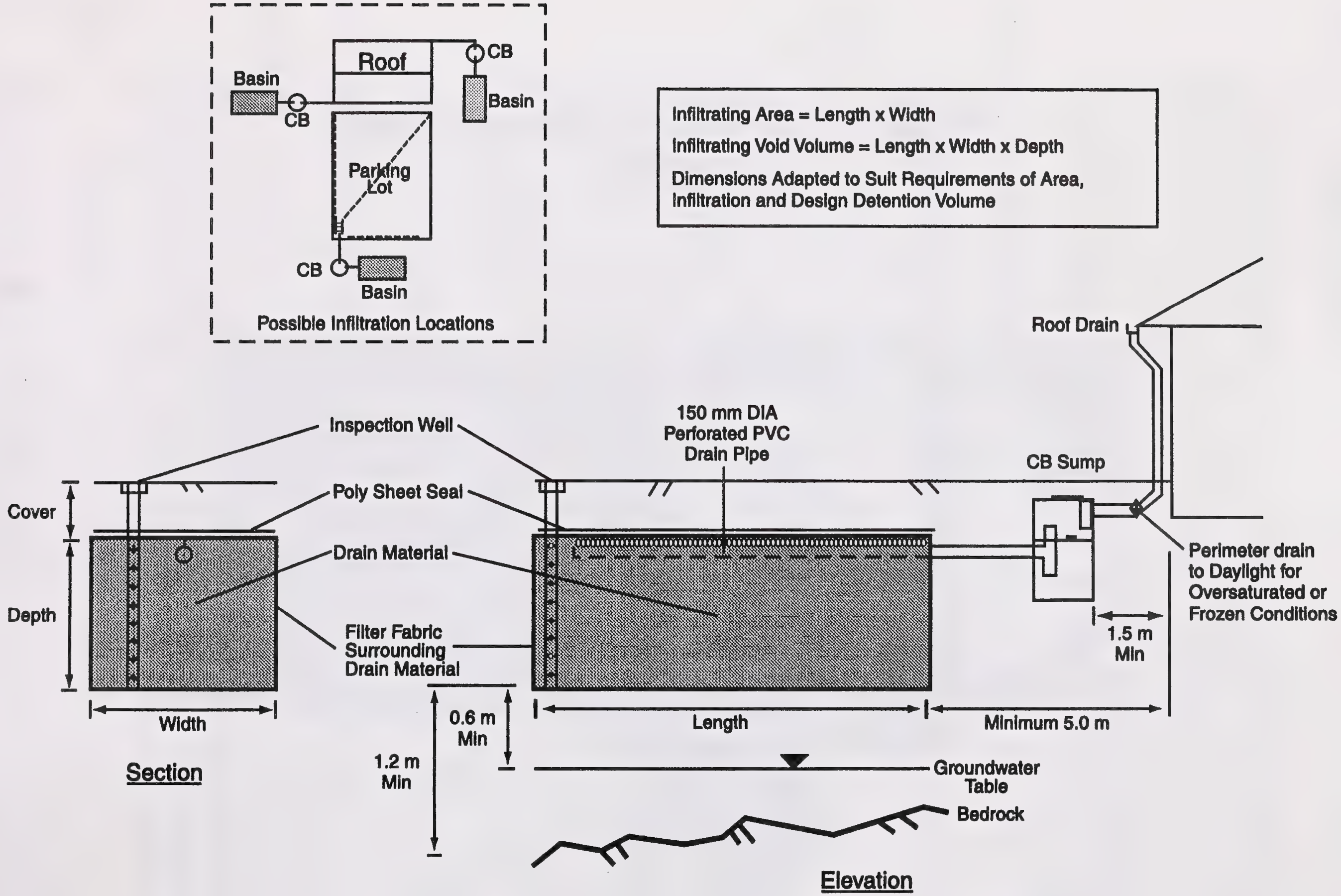


Figure 6.4
Infiltration System with Pretreatment Sump

6.3.4.3 Applicability

Sump pumps are applicable in most cases. However, excessive pumping may occur in high groundwater and high bedrock conditions if they are within 1 m of the foundation drain. Under conditions where infiltration systems are appropriate, or where overland flow paths are available, sump pumps can discharge to either the infiltration system or to the surface.

6.3.4.4 Effectiveness

Foundation drainage is normally relatively clean water and is well suited to the optimal operation of infiltration systems or overland flow to rear yard ponds.

6.3.4.5 Water Quantity

The impact of foundation drain discharge on downstream stormwater management facilities is dependent on the original discharge location. If foundation drainage was originally discharged to the storm sewer network or to the sanitary sewer, there will be some reduction in stormwater flow in the sewer. There will also be additional groundwater recharge and potentially baseflow augmentation in the local receiving stream if foundation drainage was originally discharged to either the storm sewer or sanitary sewer networks.

6.3.4.6 Water Quality

Foundation drainage is relatively clean water and if flow is removed from either the storm sewer network or the sanitary sewer network there is likely to be some impact on the dilution of contaminants provided by the foundation drainage.

6.3.4.7 Design Considerations

Sump pump drainage to an infiltration system is illustrated in Figure 6-5. The location of the infiltration system should conform to infiltration design considerations. Yard grades should conform to design considerations for infiltration ponds. Sump pump discharges should be located at least 2.0 m away from foundations and be discharged to rear yards away from sidewalks to prevent icing conditions during winter months. Discharges should also be located at least 0.5 m above ground to prevent blockage from ice and snow during the winter.

6.4 Stormwater Conveyance System BMPs

Stormwater conveyance systems transport drainage from developed areas through sewer or grassed swale systems. Stormwater conveyance controls are applied as part of the stormwater conveyance system and can be classified into three categories:

- Pervious pipe systems
- Pervious catchbasins
- Grassed swales

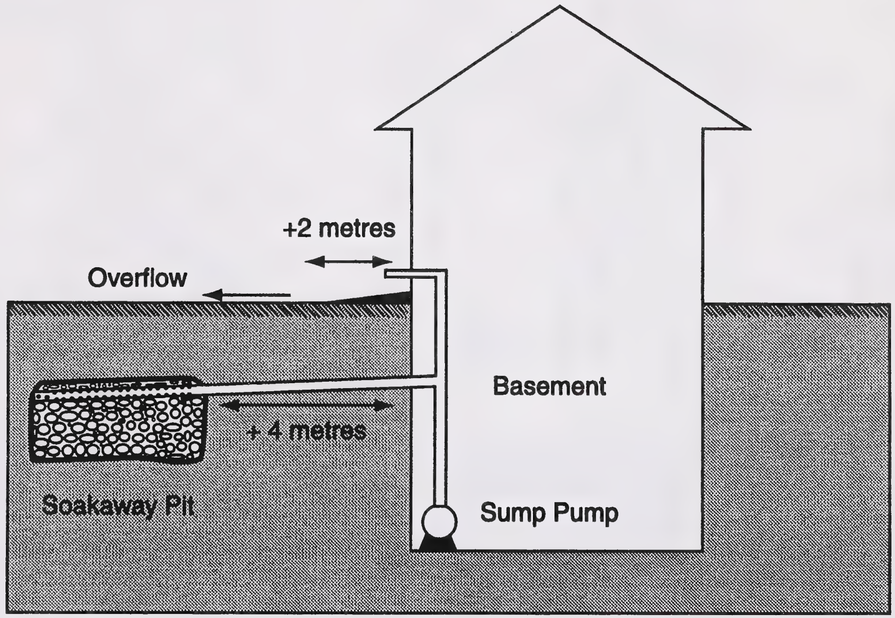


Figure 6.5
Sump Pump Foundation Drainage

6.4.1 Pervious Pipe Systems

6.4.1.1 Purpose

Pervious pipe systems are intended to convey and infiltrate drainage.

6.4.1.2 Description

Pervious pipes have not been commonly used but they are very similar to a conventional tile drainage system. Pervious pipe systems are perforated along their length, thereby promoting exfiltration of stormwater as it is conveyed downstream.

Pervious pipe networks are commonly components of roadway drainage systems. Because roadway drainage usually carries a high level of suspended sediments there are associated pretreatment components. Roadway runoff is directed toward grassed areas that act as sediment filters prior to flowing into the stormwater catchbasin. The stormwater catchbasin is raised to allow some ponding and further sediment removal. The catchbasin is connected to the pervious pipe.

6.4.1.3 Applicability

Pervious pipe systems, although being implemented in several municipalities, are still considered experimental in nature.

6.4.1.4 Effectiveness

Pervious pipe systems for the exfiltration of road runoff have not proven very reliable. Pervious pipe systems experience clogging due to the high solids loads especially during construction of the pervious pipe system in new developments. Some form of pretreatment for solids removal prior to the pervious pipe system is advisable.

6.4.1.5 Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally impervious. Any stormwater infiltrated through the pervious pipe network reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6.4.1.6 Water Quality

Road runoff normally carries high levels of solids, oils, greases, metals, and chlorides if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

Infiltration of road runoff may, however, present a groundwater contamination problem.

6.4.1.7 Design Considerations

Implementation of a pervious pipe system is illustrated in Figure 6-6. Design considerations must include the pretreatment of road runoff for solids removal. Pretreatment can be accomplished by incorporating grassed boulevards as pretreatment areas. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. The infiltration pipe must be a sufficient height above the groundwater table to prevent groundwater from flowing into the pipe and to allow for proper infiltration.

The minimum storage volume should be equal to the runoff from a 5-mm storm over the contributing drainage area. The storm volume should be accommodated in the pervious pipe bedding/storage media without overflowing. The maximum storage area should be equal to the runoff from a 25-mm storm over the contributing drainage area. The exfiltration storage bedding depth should be 75 mm to 150 mm deep above the crown of the pervious pipe and the bedding should drain within 24 hours. The minimum diameter for the pervious pipe should be 200 mm and the pipe should be smooth walled to reduce the potential for clogging

6.4.2 Pervious Catchbasins

6.4.2.1 Purpose

Pervious catchbasins are intended to convey and infiltrate road drainage.

6.4.2.2 Description

Pervious catchbasins are normal catchbasins with larger sumps that are physically connected to an exfiltration storage media. The storage media is generally located beneath or beside the catchbasin.

6.4.2.3 Applicability

Pervious catchbasins are still considered to be experimental.

6.4.2.4 Effectiveness

Maintenance requirements for pervious catchbasins are dependent on the clogging frequency of the infiltration media which can be high given the sediment load normally associated with road runoff. Pervious catchbasins are easier to construct in new developments. They can be plugged during construction to prevent solids clogging the system.

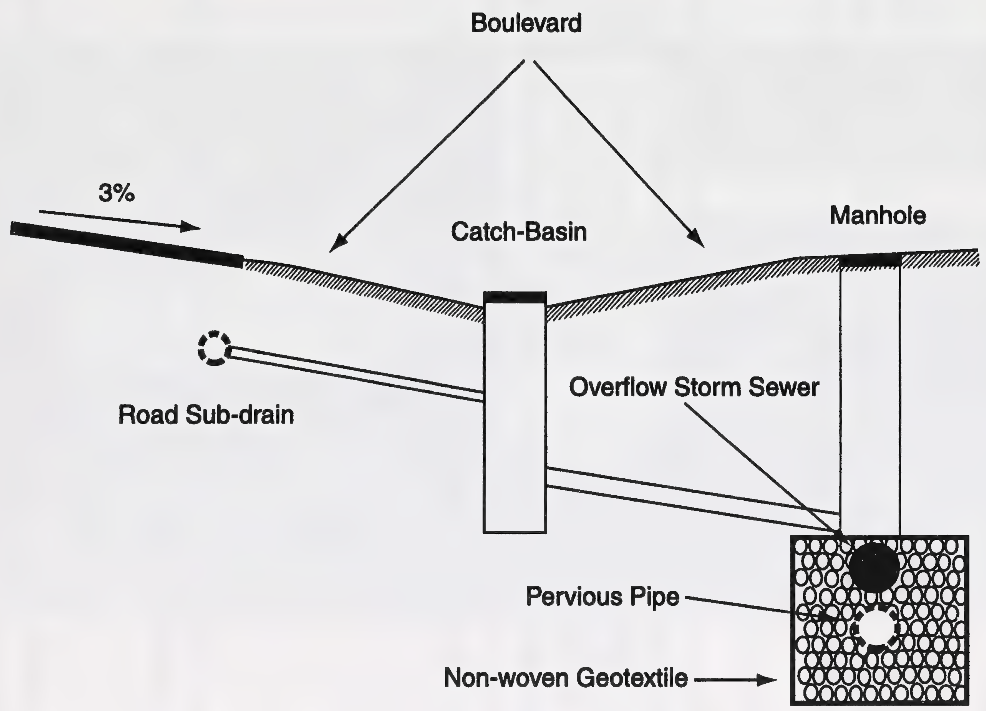


Figure 6.6
Pervious Pipe System

6.4.2.5 Water Quantity

Stormwater runoff from road surfaces contributes a substantial amount of discharge to the stormwater conveyance systems because road surfaces are normally quite impervious. Any stormwater infiltrated through pervious catchbasins reduces the total end-of-pipe discharge and therefore, any storage/treatment requirements.

6.4.2.6 Water Quality

Road runoff normally carries high levels of solids, oils, greases, and metals. Chlorides may also be a problem if road salt is applied during the winter months. Removal of these contaminants prior to end-of-pipe can enhance the performance of any storage or treatment facilities. Stormwater quality can substantially improve at the end-of-pipe discharge point.

6.4.2.7 Design Considerations

The application of a pervious catchbasin for road runoff control is illustrated in Figure 6-7. The most important design consideration is the provision of adequate pretreatment of solids to prevent frequent clogging. Design specifications recommend construction at least 1 m above the groundwater table and the use of appropriate geotextile and clear 50-mm stone to promote filtration with a low clogging frequency. To be an effective method of infiltration the surrounding soils must have a high infiltration potential. Storage volume criteria should be the same as that for pervious pipe. The depth of the exfiltration storage is dependent upon the native soil characteristics. Maximum depths can be calculated based on the native soil percolation rate. The physical dimensions of the storage will depend on the area of land available.

6.4.3 Grassed Swales

6.4.3.1 Purpose

Grassed swales store, infiltrate and convey road and on-lot stormwater runoff. Grassed swales are normally associated with low-density developed drainage basins.

6.4.3.2 Description

Grassed swales are natural depressions or wide shallow ditches. The grass or emergent vegetation in the swale acts to reduce flow velocities, prevent erosion, and filter stormwater contaminants.

6.4.3.3 Applicability

Grassed swales are typically used in more rural areas with rolling or relatively flat land but can be used in place of or as an enhancement to any stormwater curb and gutter system except in strip commercial and high-density residential areas. In rural areas and in urban applications, grassed swales have been shown to effectively infiltrate runoff and remove pollutants.

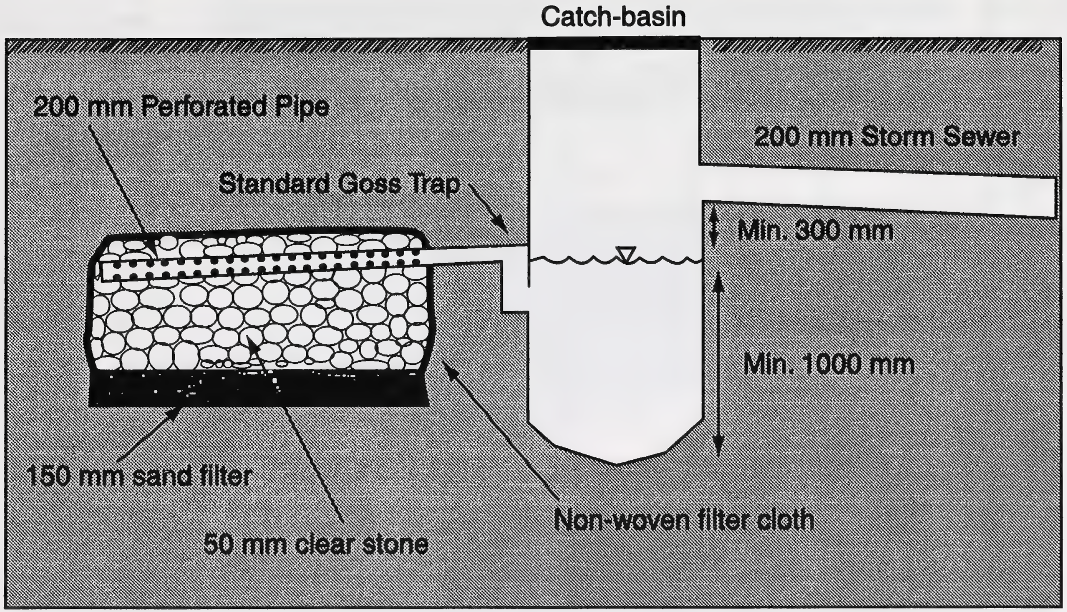


Figure 6.7
Pervious Catch-Basin

Grassed swales are being designed more frequently to replace curb and gutter controls and can be recommended for consideration in both rural and urban drainage basins.

6.4.3.4 Effectiveness

Grassed swales have been reported by many agencies in Canada and the U.S to provide effective quantity and quality control of urban and rural runoff. Grassed swales must be properly maintained to ensure effectiveness and prevent ponding of water. If water is allowed to pond in the swale, wetland vegetation may grow and mosquitos may become a problem.

6.4.3.5 Water Quantity

Grassed swales infiltrate stormwater and reduce the end-of-pipe discharge volumes normally associated with curb and gutter controls. Significant amounts (up to 95 percent) of runoff reduction are reported in the literature pertaining to grassed swales. Grassed swales also significantly lower peak discharge rates associated with frequent storms. The changes in runoff discharge volumes and rates also reduce erosion in downstream systems.

6.4.3.6 Water Quality

Grassed swales can be effective in filtering and detaining stormwater runoff from a variety of catchment types. Grassed swales are effective for stormwater treatment as long as minimum channel slope is maintained and a wide bottom width is provided. Many stormwater contaminant particulates are effectively filtered by grassed swales including heavy metals, COD, nitrate nitrogen, ammonia nitrogen, and suspended solids. Other contaminant nutrients such as organic nitrogen, phosphorus, and bacteria have been reported to bypass grass swales.

6.4.3.7 Design Considerations

General design considerations for a grassed swale are shown in Figure 6-8. An illustration of a grassed swale with a check dam is shown in Figure 6-9.

Swales should be designed with minimum longitudinal slopes (1 to 2 percent) to promote infiltration and filtering characteristics but still maintain conveyance requirements to prevent flooding and local ponding in the swale. Check dams, as shown in Figures 6-8 and 6-9, are normally used when the longitudinal slope exceeds 2 to 4 percent. Figure 6-8 shows a perforated pipe enhancement to the swale that ensures the swale remains dry between storm events. Side slopes should be no greater than 2.5 to 1 but are optimally less than 4 to 1. A minimum bottom width of 0.75 m and minimum depth of 0.5 m should be maintained. The maximum velocity in the swale should be 0.5 m/s. Where velocities are greater than 0.5 m/s the use of check dams (Figure 6-9) can promote infiltration and settling of pollutants. Grass should be local species or standard turf grass where a more manicured appearance is required. The grass should be allowed to grow higher than 75 mm so that suspended solids can be filtered effectively.

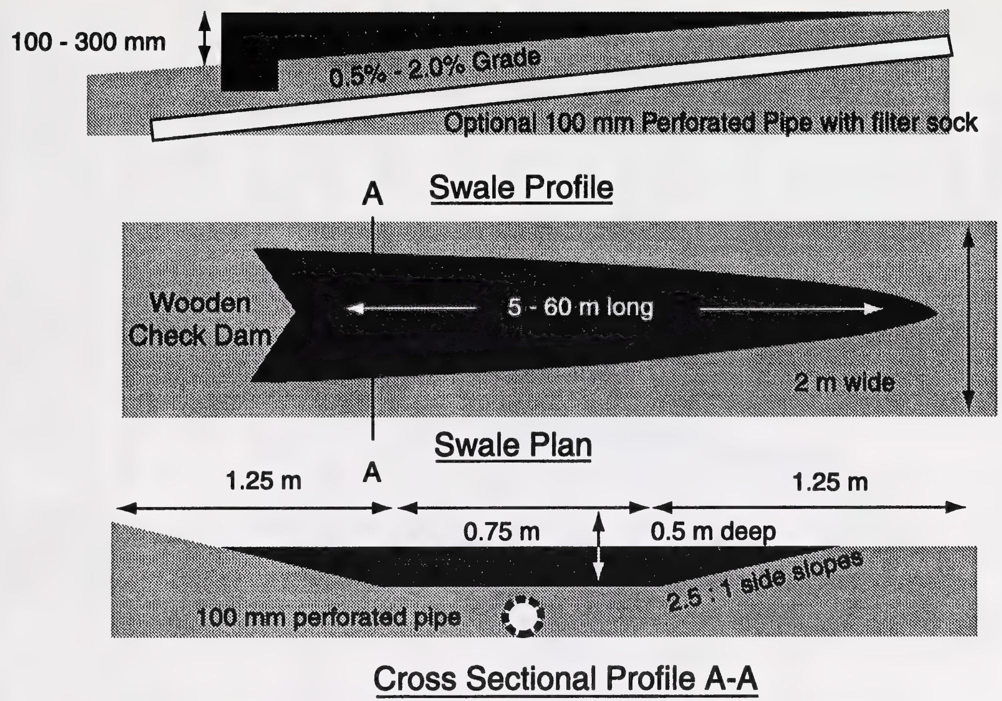


Figure 6.8
Grass Swale Design

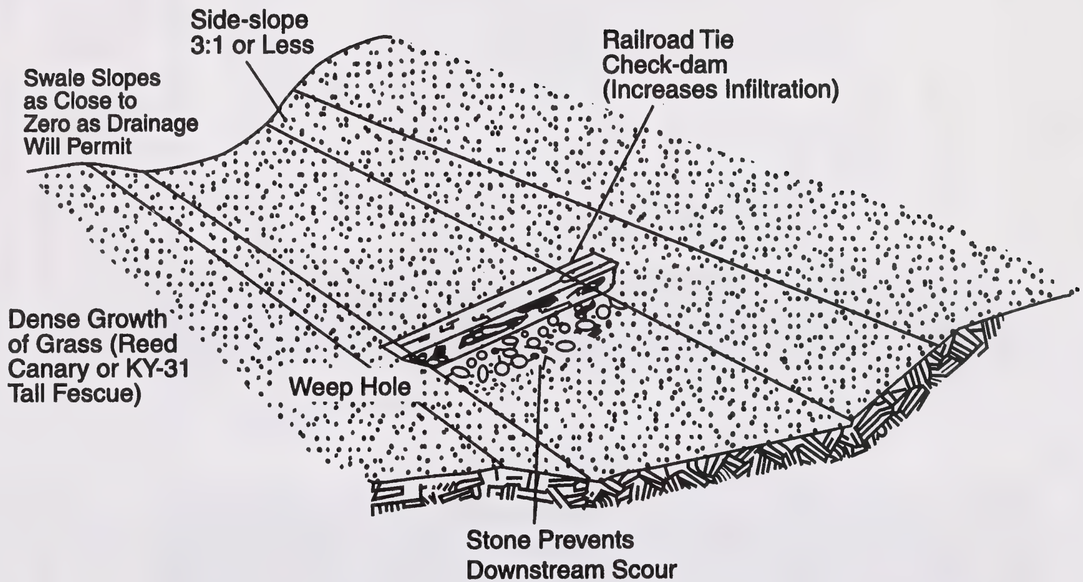


Figure 6.9
Grass Swale with Check Dam

6.5 End-of-Pipe Stormwater BMPs

End-of-pipe stormwater BMPs provide water quality enhancement to stormwater prior to discharge into a receiving water body. A number of end-of-pipe alternatives are available for application depending on the characteristics of the upstream catchment and the requirements for water quality enhancement. Eight general categories of end-of-pipe BMP facilities are discussed:

- Wet ponds
- Dry ponds
- Wetlands
- Infiltration trenches
- Infiltration basins
- Filter strips
- Sand filters
- Oil/grit separators

All references to "wet ponds", "wetlands", or "dry ponds" assumes extended detention storage is provided. Extended detention refers to the dry or active storage provided by these facilities. Extended detention ponds reduce the rate of stormwater discharge by storing the stormwater runoff temporarily and releasing it at a controlled rate. Water quality treatment is provided through enhanced settling and biological processes. As such, extended detention storage provides benefits related to water quality, erosion protection, and flooding potential.

6.5.1 Wet Ponds

6.5.1.1 Purpose

The purpose of wet ponds is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potentials.

6.5.1.2 Description

Wet ponds can be created as an impoundment by either constructing an embankment or excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. The deep, permanent pond is the wet pond's primary water quality enhancement mechanism. Runoff entering the retention basin is designed to displace water already in the permanent pool and remain there until another storm event. Runoff entering the basin is slowed by the permanent pool and suspended pollutants are allowed to settle. Biologic processes, such as nutrient uptake by algae, are established in the permanent pool and help reduce concentrations of soluble contaminants. A vegetative planting strategy should provide shading, aesthetics, safety, and enhanced pollutant removal.

6.5.1.3 Applicability

A reliable source of runoff or groundwater discharge must be available to maintain the permanent pool of a wet pond. As such, wet ponds are generally considered for drainage areas greater than 5 ha. Because of a wet pond's ability to reduce soluble pollutants, it is generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Wet ponds may not be appropriate, or may require specialized design, where receiving water temperatures are a concern.

6.5.1.4 Effectiveness

Wet ponds are probably the most common end-of-pipe management facility for the control of peak runoff discharges and the enhancement of water quality. Wet ponds are very effective in controlling runoff and improving water quality when proper design considerations are made for those two objectives.

6.5.1.5 Water Quantity

As a detention facility, a wet pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Wet ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Wet ponds are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wet ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a pre-determined erosive threshold.

6.5.1.6 Water Quality

Wet ponds have been cited as providing the most reliable end-of-pipe BMP in terms of water quality treatment. This reliability is attributed to a number of factors including:

- Performance does not depend on soil characteristics
- Permanent pool prevents re-suspension
- Permanent pool minimizes blockage of outlet
- Promotes biological removal of pollutants
- Permanent pool provides extended settling

Wet ponds have a moderate to high capacity to remove most urban pollutants depending on how large the volume of the permanent pool is in relation to the runoff produced from the contributing drainage area. The establishment of vegetative zones in and around a wet pond can enhance its pollutant removal capability.

6.5.1.7 Design Considerations

Wet ponds must be designed to meet specific water quality and/or discharge rate objectives. Wet ponds designed to control peak discharge rates do not normally provide optimum water quality enhancement. Flood control or peak flow control wet ponds are typically designed to control the large infrequent event storms. Water quality wet ponds need to be designed to capture and treat the more frequent smaller storms with which the majority of the contaminant loadings are associated. Wet ponds can be designed to meet both flood control and water quality objectives.

One of the primary criteria for the proper design of a wet pond for peak runoff control is the provision of adequate detention storage volume. The primary design consideration for a wet pond for water quality enhancement is the settling velocity of the particulates in the stormwater entering the pond. The wet pond surface area is directly related to this required settling velocity. Ponds designed only for peak flow reduction do not normally provide adequate facility for water quality enhancement.

The design of a wet pond requires careful consideration of the required design objectives for flood control and water quality enhancement. Figure 6-10 illustrates some of the basic recommendations for a wet pond. Detailed designed requirements should be evaluated for each individual application based on site specific constraints and objectives.

Some general design parameters are:

- Minimum water surface area of 2 ha
- Maximum sideslopes above active storage zone are 4:1 to 5:1
- Maximum interior sideslopes in active storage zone are 5:1 to 7:1
- Maximum exterior sideslopes are 3:1

Some water quality control design parameters are:

- Permanent pool sized to store the volume of runoff from a 25-mm storm over the contributing area
- Detention time of 24 hours
- Length to width ratio shall be from 4:1 to 5:1
- Minimum permanent pool depth of 2.0 m
- Maximum permanent pool depth of 3.0 m The maximum water level should be below adjacent house basement footings.

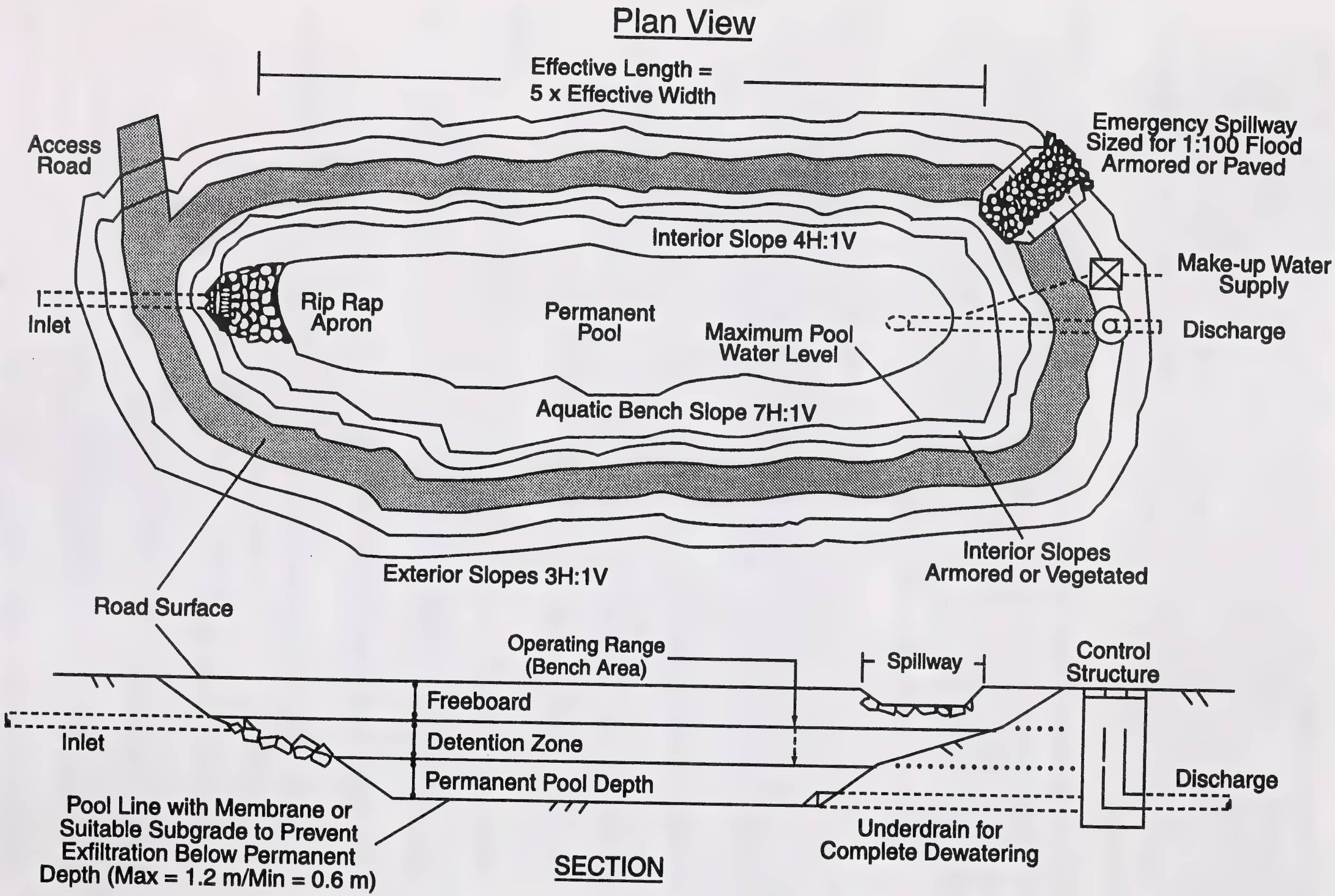


Figure 6.10
Wet Detention Pond Plan and Sections

- Maximum active detention storage depth of 2.0 m

Some water quantity control design parameters are:

- 1-in-100-year storm stored within 2 m above the permanent pool (Alternatively, the 2 m can be used to store the 1-in-25-year storm. In such cases an emergency overflow drainage system should be constructed with the capacity to carry storm runoff from the 1-in-100-year storm event to receiving streams or downstream stormwater management facilities.)

- Detention time of 24 hours

Also, wet pond water quality control performance can be improved by providing a pretreatment sump or forebay and a backup water supply to maintain the minimum storage volume. During the design process, other design considerations should be evaluated that relate to ease of maintenance. The forebay should be designed with the following parameters:

- Length to width ratio of 2:1 or greater

- Forebay surface area not to exceed one-third of the permanent pool surface area

- Forebay length, L_{fb} as follows:

$$L_{fb} = [rQ_p/V_s]^{0.5}$$

where
r = Length to width ratio of forebay
Q_p = Peak flow rate from the pond during the design quality storm (m^3/s)
V_s = Settling velocity (dependent on the desired particle size to settle)

- Dispersion length, L_{dis} as follows:

$$L_{dis} = (8Q)/(dV_f)$$

where
Q = inlet flow rate (m^3/s)
d = depth of permanent pool in the forebay (m)
V_f = desired velocity at the end of the forebay

- Forebay Bottom Width, W = $\mathbf{L_{dis}/8}$

- Forebay berm should be 0.15 to 0.3 metres below the permanent pool elevation. Forebays can be constructed with a solid substrate to facilitate removal of accumulated sediment and debris.

6.5.2 Dry Ponds

6.5.2.1 Purpose

The purpose of a dry pond is to temporarily store stormwater runoff in order to promote the settlement of runoff pollutants and to restrict discharge to predetermined levels to reduce downstream flooding and erosion potential.

6.5.2.2 Description

Dry ponds are impoundment areas constructed by an embankment or through excavating a pit. They are often designed as a two-stage (dual-purpose) facility, where the upper stage (flood fringe area) is designed to store large, infrequent storms, and the lower stage (extended detention stage) is designed to store, and promote sedimentation, of smaller, more frequent storms. However, unlike wet ponds, the lower stage is designed to empty completely between storm events.

6.5.2.3 Applicability

Dry ponds may be applied where topographical or planning constraints exist that limit the land available for wet ponds. Drainage areas greater than 5 ha are generally recommended for dry ponds. The use of dry ponds for combined water quantity and quality control is discouraged without the use of sediment forebays that include a permanent pool.

A dry pond's limited effectiveness in removing soluble contaminants is an important factor in considering its application. For example, in low-density residential areas where soluble nutrients from fertilizers and pesticides are a concern, dry ponds in isolation may not be appropriate.

6.5.2.4 Effectiveness

Dry ponds do not provide water quality enhancement because of the bottom scour that occurs with each storm event. Dry ponds do provide effective stormwater flow attenuation.

6.5.2.5 Water Quantity

As a detention facility, a dry pond typically flattens and spreads the inflow hygrograph, thus lowering the peak discharge. Dry ponds are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond release rates for regional flood control. Dry ponds are relatively ineffective for volume reduction, although some evaporation may occur. Dry ponds are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6.5.2.6 Water Quality

Because dry ponds have no permanent pool of water, the removal of stormwater contaminants in dry ponds is a function of the pond's drawdown time. The removal of soluble pollutants does not generally occur in a dry pond. Without a permanent pool, re-suspension of contaminants is a concern. Dry ponds operating in a continuous mode are generally less effective at pollutant removal compared to wet ponds, whereas dry ponds operating in a batch mode have been reported to be similarly effective. In general, dry ponds should only be implemented if it is determined that a wet pond cannot be implemented due to topographical or planning constraints.

6.5.2.7 Design Considerations

The design of a dry pond has many site-specific requirements that must be considered. These design considerations are dependent on the constraints of a particular site and the objectives for the pond.

Figure 6-11 illustrates some of the basic recommendations for a dry pond.

Some general design parameters are:

- Storage capacity for up to the 1-in-100-year storm
- Detention time of 24 hours
- Maximum active retention storage depth of 1.0 to 1.5 metres. The maximum water level should be below adjacent house basement footings.
- Maximum interior sideslopes of 4:1 to 5:1
- Maximum exterior sideslopes of 3:1
- Minimum freeboard of 0.6 m
- Minimum ratio of effective length to effective width of 4:1 to 5:1
- Minimum slope in the bottom of the pond of 1 percent (2 percent is preferred)

During the design process, other design considerations should be evaluated that relate to ease of maintenance and use. For example, a weeping tile system could be installed under the bottom of the pond to improve the rate at which the pond bottom dries out between storm events.

6.5.3 Constructed Wetlands

6.5.3.1 Purpose

By retaining runoff for a prolonged period of time and uptaking, altering, and storing pollutants, constructed wetlands serve to improve water quality and control peak discharge rates.

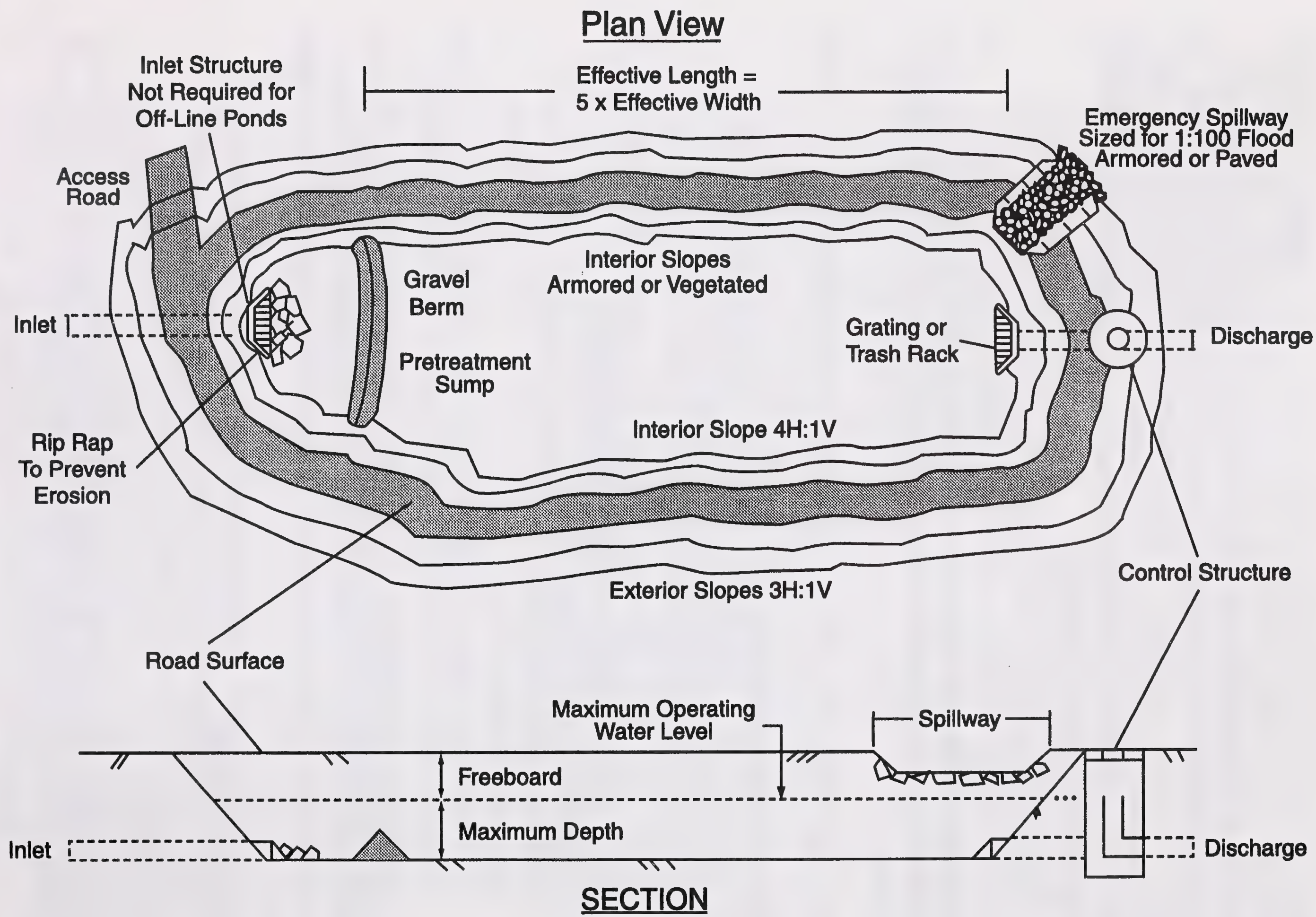


Figure 6.11
Dry Detention Pond Plan and Sections

6.5.3.2 Description

There are five basic stormwater wetland designs: shallow marsh, pond/wetland, extended detention wetland, pocket wetland, and fringe wetland. All are essentially surface flow systems, with varying emergent marsh and deep pool habitat, hydraulic capacity, residence time, and travel routes.

Constructed wetlands can be created as an impoundment by either constructing an embankment or excavating a pit. Relatively deep permanent pools are maintained at the inlet and outlet and along low flow paths to minimize the re-suspension and discharge of settled pollutants from the facility. Relatively shallow extended detention storage areas with extensive plantings (submergent and emergent) make up the majority of a constructed/artificial wetland's permanent storage. Sedimentation, filtration and biological processes account for the water quality benefits afforded by wetlands. Planting strategies are also implemented for shoreline fringe areas and/or floodfringe areas (if a combined facility) providing shading, aesthetics, safety, and enhanced pollutant removal.

6.5.3.3 Applicability

In recent years, interest has shifted from providing stormwater attenuation with retention ponds alone, to incorporating vegetated wetland cells into the design to provide greater attenuation and contaminant removal and improved landscape aesthetics in urban environments. Many communities across Canada have installed wetlands as part of their stormwater management systems. Several additional installations are awaiting approval from the regulatory authorities and there are many others in the predesign or design phase.

Generally wetlands can be considered for drainage areas greater than 5 ha Because of a wetlands ability to reduce soluble pollutants, they are generally applicable to residential, commercial, or industrial areas where nutrient loadings may be expected to be relatively high. Constructed/artificial wetlands may not be appropriate, or may require specialized design, where receiving-water temperatures are a concern. The application of constructed/artificial wetlands may be further constrained by existing planning designations or topography that limits land availability. Potential ancillary benefits provided by wetlands include aviary, terrestrial, and aquatic habitat.

Wetland water treatment systems are not recommended for all applications. Such systems are most appropriate under the following conditions:

- Large tracts of suitable land are readily available.
- The influent does not contain high levels of industrial toxic pollutants as identified by provincial and federal agencies.
- There is a shortage of local groundwater or surface water supplies.
- A water body with impaired water quality is located in the area.
- The region has a history of wetland loss.
- Regulatory agencies are interested in the potential benefits of the technology.

A study carried out in 1994 for the North American Wetland Conservation Council (Canada) to determine the extent to which constructed wetlands were being used in Canada for stormwater and wastewater treatment. After contacting more than 100 people who had some involvement with natural treatment systems, 67 wetland treatment systems were identified. Of these, 10 were stormwater treatment wetlands.

In Alberta, the City of Calgary has set up a task force sanctioned by Alberta Environment to investigate the feasibility of using wetlands for stormwater treatment. Numerous potential sites have been identified and it is anticipated that of the eight to 10 sites to be chosen, two sites will be highly monitored. One site treating runoff from a farming practice was using a slough consolidation and runoff retention method.

6.5.3.4 Effectiveness

Stormwater wetland water treatment systems provide several major benefits:

- They require less maintenance and are less expensive to maintain than traditional treatment system.

- With proper design, portions of the wetland treatment system may provide additional wetland wildlife habitat, as well as recreational opportunities such as bird watching, hiking, and picnicking.

- Wetland treatment systems are viewed as an asset by provincial and federal agencies in many regions and as a potentially effective method for replacing wetlands lost through agricultural practices, industrial and municipal development, and groundwater withdrawal. These systems may provide mitigation banking, if this practice is adopted in Canada as it has been in the U.S., for future planned use of low-value wetland areas.

6.5.3.5 Water Quantity

Wetlands typically flatten and spread the inflow hygrograph, thus lowering peak discharges. Wetlands are effective in controlling the post-development peak discharge rate to the desired pre-development levels for design storms. Watershed/subwatershed analyses should be performed to coordinate subcatchment/pond/wetlands release rates for regional flood control. Wetlands are relatively ineffective for volume reduction, although some infiltration and/or evaporation may occur. Wetlands are generally effective in controlling downstream erosion if designed such that the duration of post-development "critical impulses" does not exceed a predetermined erosive threshold.

6.5.3.6 Water Quality

In general, wetland water treatment systems have been found to lower BOD, TSS, and total nitrogen concentrations to 10 to 20 percent of the concentrations entering the systems. For total phosphorus, metals, and organic compounds, removal efficiencies vary widely, typically

from 20 to 90 percent. Removal of these latter constituents appears to be limited by substrate type, the form of the constituents, the presence of oxygen, and the entire chemical makeup of the water to be treated.

The mechanisms for treating urban stormwater are both chemical and physical and include:

- Volatilization. Many pollutants may be dispersed by evaporating. They include oils, mercury, and some chlorinated hydrocarbons.

- Sedimentation. This is a principal mechanism for removal of particulate pollutants. The deposition is affected by flow rates, paths and storm size. Sedimentation is important also in removing suspended solids, particulate nitrogen, oils, chlorinated hydrocarbons, and most heavy metals.

- Adsorption. Dissolved pollutants adhere to suspended solids that settle out. It is also the primary-viral removal mechanism.

- Precipitation. Many ions precipitate in response to changes in pH, oxygen concentration, etc. in wetlands.

- Filtration. Many particulate pollutants may be filtered through the vegetation and soils of wetlands.

6.5.3.7 Design Considerations

The design of a constructed wetland for dealing with urban stormwater requires a detailed study to determine from the outset what the goals of the wetland are. If the function is primarily to store water during storm events and release it later, then the size of the catchment area, permeability of the urban surfaces, and recorded flow rates will be used to determine the water volume storage capacity required. This, together with the expected frequency of large storm events, will provide an indication of the suggested drawdown rates for the wetland and the diameter of outflow pipes. If, on the other hand, improving water quality is a major goal, then subsurface water flow through one or more cells may be worth incorporating into the design specifications. Should the wetland operate in the fall, winter, and early spring as well as in summer? If so, then a configuration of wetland that is deep and permits water flow during low winter temperatures may be appropriate.

Several goals may be identified for a constructed wetland, but the available site may limit the achievement of all the goals. In this case priorities must be set. The general location of a constructed wetland is an important consideration. Is it to be constructed in a residential, industrial, or rural area? Considerations such as safety, aesthetics, potential toxic spills, or wildlife mean that different design criteria must be considered. To achieve water management goals, social as well as technical issues must be addressed, for "social" problems may be more difficult to solve than physical and technical ones, and managers should involve local interest groups in the early planning stages of projects.

It is important that a pretreatment area be provided for the collection of sediment and for the protection of the constructed wetland from accidental spills. Construction of a pretreatment area for oil separation and sediment removal prior to allowing water to flow into a wetland is recommended.

A constructed wetland could contain a number of cells, either of similar construction and function, or of different structure and purpose. Figure 6-12 illustrates the major components of a constructed wetland.

General design considerations are as follows:

- Design and implement with designated objectives constantly and clearly in mind.

- Design more for function than for form. A number of forms can probably meet the objectives, and the form to which the system evolves may not be the planned one.

- Design relative to the natural reference system(s), and do not over-engineer.

- Design with the landscape, not against it. Take advantage of natural topography, drainage patterns, etc.

- Design the wetland as an ecotone. Incorporate as much "edge" as possible, and design in conjunction with a buffer and the surrounding land and aquatic systems.

- Design to protect the wetland from any potential high flows and sediment loads.

- Design to avoid secondary environmental and community impacts.

- Plan on enough time for the system to develop before it must satisfy the objectives. Attempts to short-circuit ecological processes by over-management will probably fail.

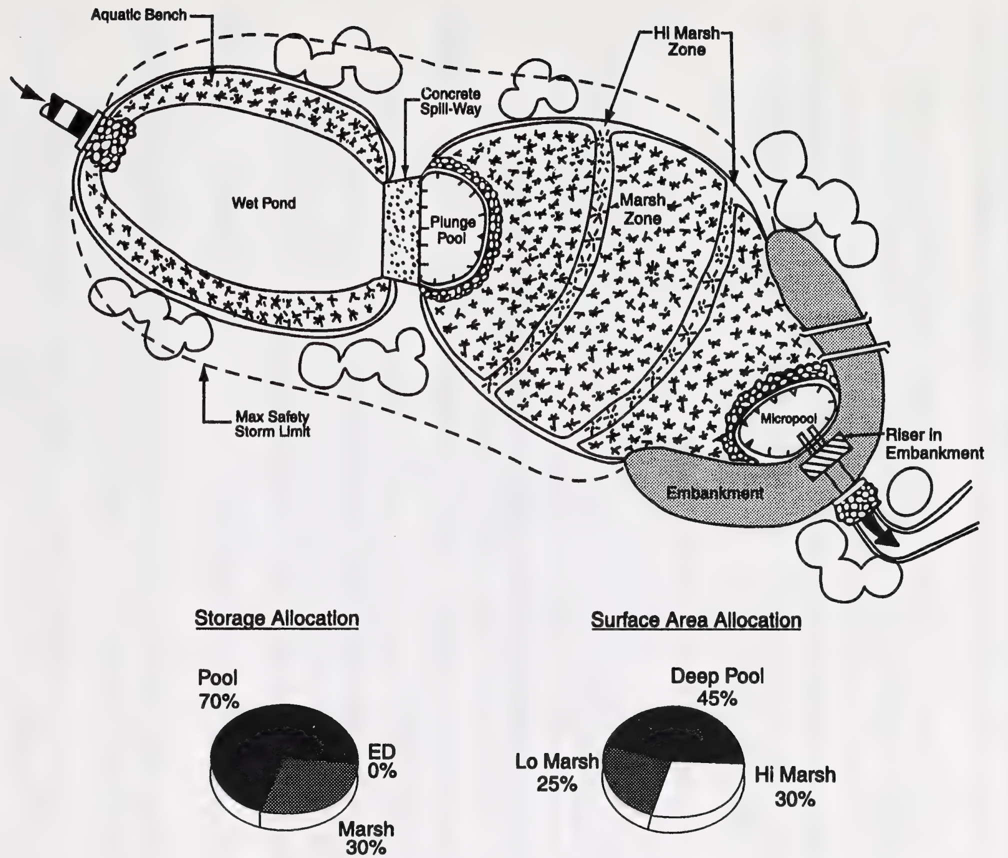
- Design for self sustainability and to minimize maintenance.

Considerations for the size and configuration of the wetland are:

- Wetland size should be approximately 5 percent of the watershed area that it will be servicing

- Approximately 10 percent of the wetland surface area should be a 1.5 to 20.0 m deep forebay upstream of the wetland area for settleable solids removal

- Average active water depth of 0.3 m (below any ice coverage allowance) with 1 m deep zones for flow redistribution and deeper for fish and submerged or floating aquatic vegetation habitat. Freezing of the wetland will not affect the plants and



The pond/wetland system consists of two separate cells - a deep pond leading to a shallow wetland. The pond removes pollutants, and reduces the space required for the system.

Figure 6.12
Stormwater Wetland

vegetation. The efficiency will be much lower. Fish restocking will likely be required unless deep zones are increased to ensure survival. However, this will add to the construction cost.

- Length to width ratios can be as low as 1:1

- Shape of the treatment cell(s) can vary and depends on landscaping features required for attracting wildlife and for public enjoyment, and shape of available land

- Bottom slope of 0.5 to 1.0 percent is recommended. A flat bottom promotes sheet flow through the system

- Vegetation can be cost effectively transplanted from local donor sites including ditches maintained by the Province and construction sites where small pocket wetlands are to be removed

Flow regime and control recommendations as follows:

- Gravity flow is the preferred method of movement of water into, through and out of the treatment wetland

- Divert high flows during extreme rainfall events around the wetland

- Inflow and outflow structures that will accommodate a wide range of rainfall intensities are required.

Ancillary benefits that increase the value of the wetland are:

- Landscaped features may provide a park-like setting

Nuisance controls that should be considered are:

- Mosquito control which includes introducing or making habitat available for baitfish (fathead minnows), dragon flies, purple martins, swallows, and bats

- Odour control is not usually required since treatment wetlands, if designed properly, do not generate odours

- Nuisance wildlife, including carp and muskrat, will require control since they will destroy or consume the wetland vegetation and will, in the case of carp, re-suspend settled materials

- Freezing conditions during the winter months will not adversely affect the wetland community (plants, microbes) but treatment efficiency will be reduced. Also, during

spring runoff, treatment for solubles will be less effective. To minimize the flushing effects of spring runoff, the wetland should be designed to avoid re-suspension of sediments. This can be accomplished through sizing, layout and planting within the wetland.

6.5.4 Infiltration Trenches

6.5.4.1 Purpose

The purpose of an infiltration trench is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration. The three basic trench systems are complete exfiltration, partial exfiltration, and water quality exfiltration. Each system is defined by the volume of annual runoff diverted to the trench and the degree to which the runoff is exfiltrated into the soils. Infiltration trenches differ from on-lot infiltration systems in that they are generally constructed to manage stormwater flow from a number of lots in a developed area, not a single property.

6.5.4.2 Description

Infiltration trenches can be constructed at ground surface level to intercept overland flow directly, or constructed as a subsurface component of a storm sewer system. Infiltration trenches are generally composed of a clear stone storage layer and a sand or peat filter layer. There are other options for the type of filter used such as a non-woven filter fabric.

6.5.4.3 Application

Infiltration trenches are best utilized as recharge devices for compact residential developments (< 2 ha), rather than as a larger-scale, water quality treatment technique. Normally, infiltration trenches are not used in commercial or industrial areas because of the potential for high-contaminant loads or spills that may result in groundwater contamination.

6.5.4.4 Effectiveness

Infiltration trenches are effective in managing runoff from small residential areas. They are also effective when constructed under grassed swales to increase the infiltration potential of the swale. Clogging of the filter material can be a frequent problem if solids inputs are high and no pretreatment in the form of grassed filter strip for surface trenches or a suitable oil/grit separator for subsurface trenches is employed. Groundwater mounding may also become a problem if infiltration volumes are too high.

6.5.4.5 Water Quantity

Infiltration trenches provide marginal water quantity control. As such, the application of infiltration trenches is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

Infiltration trenches limit the volumes of runoff normally directed toward minor drainage systems. On-lot drainage rates are also reduced. This will reduce the requirements for end-of-pipe detention storage. Effective on-lot drainage reductions on a subdivision basis will lower and flatten the receiving water inflow hydrograph.

Increased infiltration of stormwater from infiltration trenches also provides recharge to the local groundwater that may in turn discharge to local streams, thus enhancing baseflows.

6.5.4.6 `Water Quality

Pretreatment BMPs such as filter strips or oil/water separators are often used in combination with infiltration trenches to minimize the potential for suspended sediments to clog the trench. Infiltration trenches limit the volumes of runoff from smaller storm events that are normally the major contributor of receiving water contaminants. Potential contamination of groundwater should be considered when examining runoff quality directed to the infiltration trench.

6.5.4.7 Design Considerations

A surface infiltration trench and a subsurface infiltration trench are shown in Figures 6-13 and 6-14, respectively. Infiltration trenches ideally have groundwater levels and bedrock layers to be at least 1 m below the bottom of the infiltration trench. Soils must have a percolation rate of more that 15 mm/hr. A suitable filter fabric should be used to protect the stone storage media from clogging.

Careful consideration should be given to the volume of stormwater directed to the infiltration trench. Only sufficient volumes should be directed to the trench to allow, at a maximum, a forty eight hour drawdown period.

In a subsurface trench, a series of perforated pipes carries stormwater to the trench. A bypass pipe or flow path should be provided for flows in excess of the design capacity of the trench.

6.5.5 Infiltration Basins

6.5.5.1 Purpose

The purpose of an infiltration basin is to collect and provide temporary storage of surface runoff for a specific design frequency storm and to promote subsequent infiltration.

6.5.5.2 Description

Infiltration basins are above-ground pond impoundment systems that promote recharge. Water percolating through an infiltration basin either recharges the groundwater system or is collected by an underground perforated pipe system and discharged at a downstream outlet. The appearance of an infiltration basin is similar to that of a wet or dry pond.

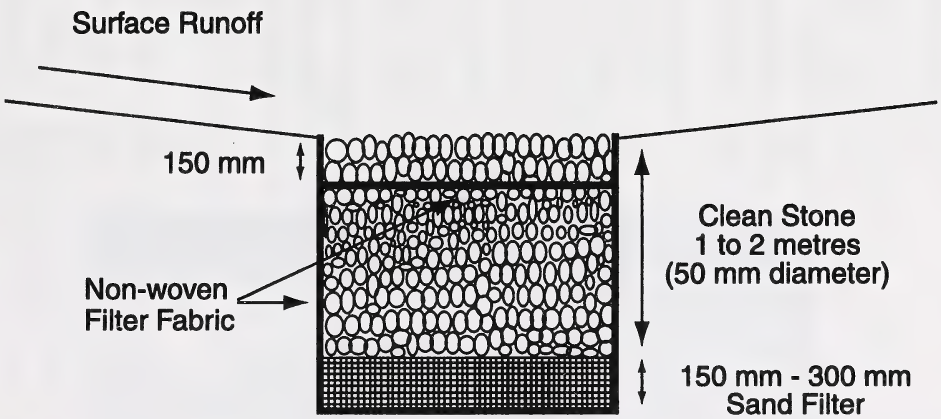


Figure 6.13
Surface Infiltration Trench

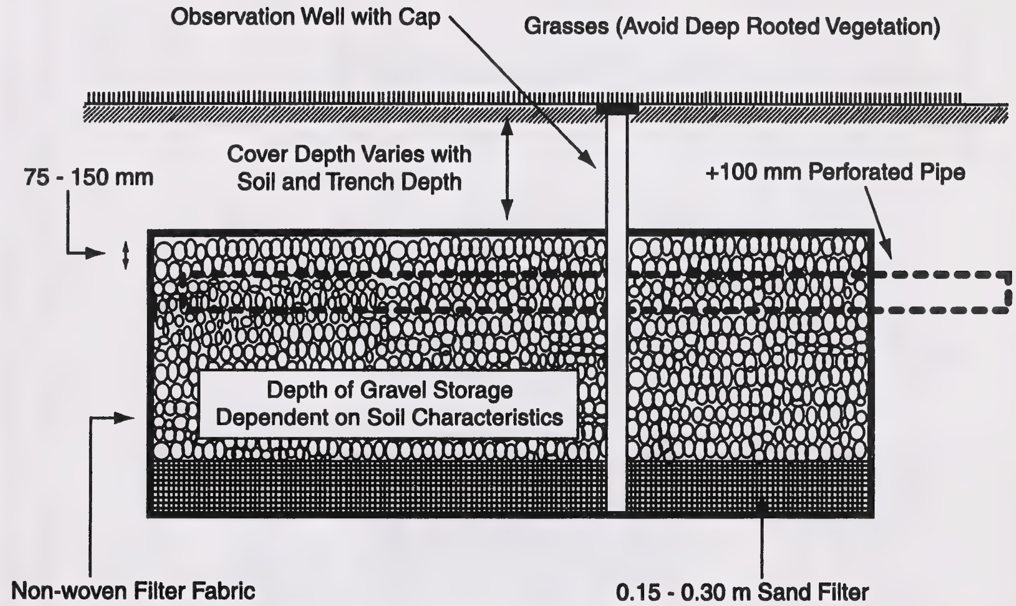


Figure 6.14
Subsurface Infiltration Trench

6.5.5.3 Applicability

Infiltration basins are generally considered for drainage areas less than 5 ha that have permeable soils. As with wet or dry ponds, an infiltration basin can be designed as a multi-stage facility to achieve various stormwater management objectives. Infiltration basins should be used in residential areas only.

6.5.5.4 Effectiveness

Infiltration basins have a very high rate of failure. Most failures can be attributed to poor site selection, poor design, poor construction techniques, large drainage area, and lack of maintenance. One of the main problems inherent in infiltration basins is that large volumes of water from a large catchment area are expected to infiltrate over a very small surface area. This leads to numerous problems and general failure of these basins.

6.5.5.5 Water Quantity

Infiltration basins are generally ineffective for water quantity control. They only infiltrate limited volumes of water from generally large catchment areas and must be provided with an overflow structure to discharge excess flow. As such, the application of infiltration basins is likely only appropriate as a secondary facility where the maintenance of groundwater recharge is a concern.

6.5.5.6 Water Quality

The application of pretreatment to reduce sediment loadings and a bypass to restrict flows during certain periods (road sanding/salting, local excavation works, facility maintenance) is recommended to improve long-term infiltration basin performance.

6.5.5.7 Design Considerations

A typical infiltration basin is illustrated in Figure 6-15. Infiltration basin design considerations must include provision for construction at the end of the development construction. Also, compaction of the basin and smearing of the basin native material must be avoided. The basin must be constructed with a maximum water storage depth of 0.6 m to avoid compaction, and the groundwater table should be a minimum of 1.0 m below the infiltration layer. Any area bedrock should also be a minimum of 1.0 m below the infiltration layer. Planting in the basin should include grasses and legumes to maintain or enhance the pore spaces in the soil.

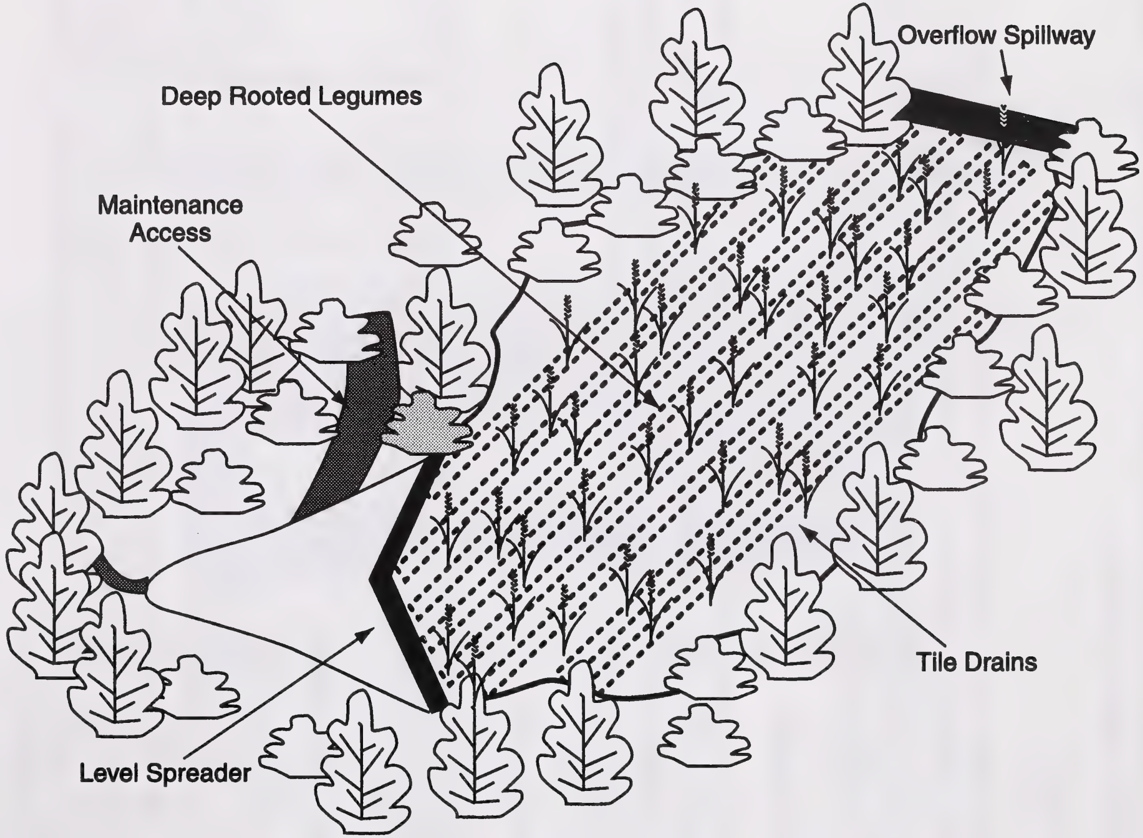


Figure 6.15
Infiltration Basin

6.5.6 Filter Strips

6.5.6.1 Purpose

Filter strips are engineered conveyance systems that are designed to remove pollutants from overland runoff. By reducing overland flow velocities, the time of concentration and infiltration are increased, thereby slightly reducing the volume of runoff and minimally controlling discharge rates.

6.5.6.2 Description

There are two general types of filter strips: grass and forested. Both consist of a level spreader, which ensures level flows, and abundant vegetative plantings. The vegetative plantings promote pollutant filtration and infiltration of stormwater. Filter strips are generally best implemented adjacent to a buffer strip, watercourse, or drainage swale, as discharge from a filter strips will be a sheet flow and thus difficult to convey in a traditional stormwater conveyance system.

6.5.6.3 Applicability

Filter strips are best applied as one of a combination of BMPs as the maintenance of sheet flow through the vegetation, and thus consistent water quality benefits, has been difficult to maintain in practice.

6.5.6.4 Effectiveness

Limited filter strip performance data are available in the literature although it is generally thought that properly designed filter strips are capable of removing a high percentage of stormwater particulates.

6.5.6.5 Water Quantity

Filter strips may slightly reduce the volume of runoff by inducing infiltration.

6.5.6.6 Water Quality

Although filter strips have been shown to be somewhat effective in removing sediment and pollutant loads in urban stormwater runoff, the ability to maintain sheet flow through the vegetation over the long term has been questioned.

6.5.6.7 Design Considerations

A schematic of a grassed and wooded filter strip is shown in Figure 6-16. The filter strip requires a level spreader with available upstream storage to regulate the discharge rate and

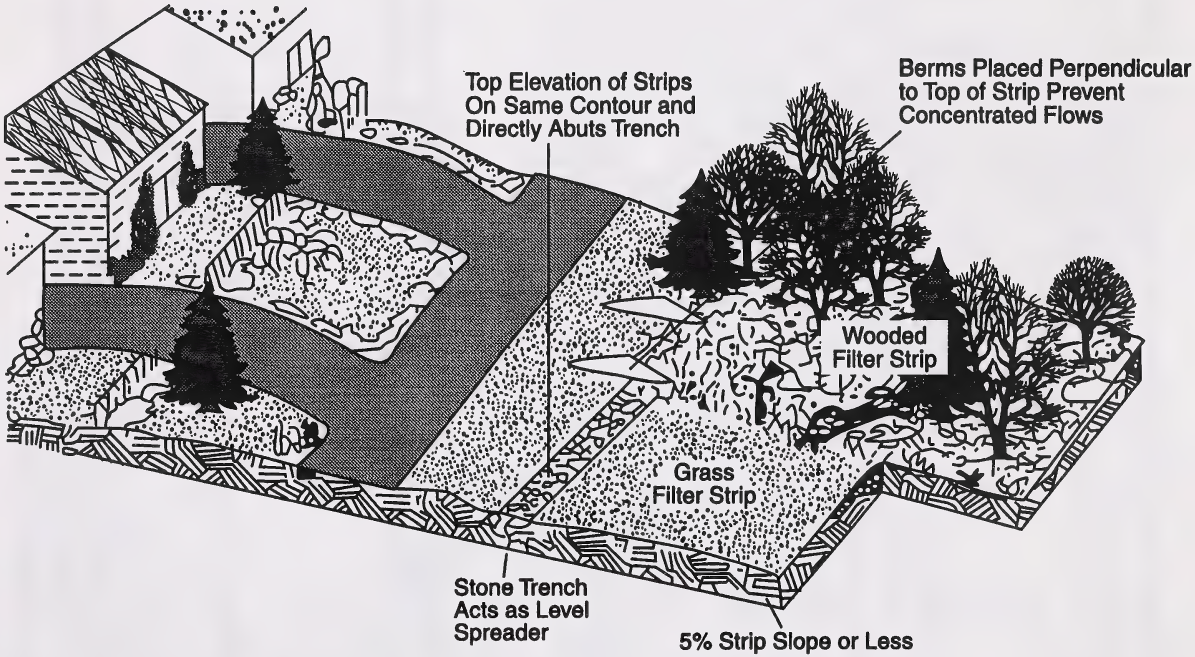


Figure 6.16
Schematic of Grassed and Wooded Filter Strip

depth of flow through the filter strip. The ideal slope for a filter strip is less than 5.0 percent over a distance of 10 to 20 m in the direction of flow.

6.5.7 Sand Filters

6.5.7.1 Purpose

Sand filters are above or below ground end-of-pipe treatment devices that promote pollutant removal from overland runoff or storm sewer systems. Sand filters do not provide a recharge benefit as filtered stormwater is discharged to the storm sewer or receiving water.

6.5.7.2 Description

Sand filters can be constructed either above or below ground as an end-of-pipe BMP. They are most commonly constructed with impermeable liners to guard against native material clogging pore spaces and to prevent filtered water from entering the groundwater system. Water that infiltrates through the filter is collected by a pervious pipe system and conveyed to a downstream outlet. Some designs incorporate a layer of peat to enhance pollutant removal capabilities of the sand filter, thus making discharge to an infiltration trench a possibility.

6.5.7.3 Applicability

Sand filters can be constructed either above or below ground as an end-of-pipe BMP and are generally only appropriate for relatively small drainage areas (< 5 ha). Also, very little is known in regard to sand filter performance and cold-climate operation and maintenance.

6.5.7.4 Effectiveness

Sand filters are not widely used in North America. This method of water quality enhancement should not be generally applied without a detailed feasibility assessment.

6.5.7.5 Water Quantity

Sand filters are not suitable for water quantity control as they should not be designed to handle large influent flows.

6.5.7.6 Water Quality

Sand filters have been found to be effective in removing pollutants in some jurisdictions, however, little is know about their performance in winter or freshet conditions.

6.5.7.7 Design Considerations

A sand filter application is illustrated in Figure 6-17. Sand filters can be constructed as surface filters or subsurface filters as part of the stormwater conveyance system. Surface filters are normally covered by a grass layer. Filters are lined with impermeable membranes to restrict clogging of the filter material by native material.

6.5.8 Oil/Grit Separators

6.5.8.1 Purpose

Oil/grit separators are a variation of traditional settling tanks. They are designed to capture sediment and trap hydrocarbons suspended in runoff from impervious surfaces as the runoff is conveyed through a storm sewer network.

6.5.8.2 Description

Oil/grit separator is a below ground, pre-cast concrete structure that takes the place of a conventional manhole in a storm drain system. The separator implements the use of permanent pool storage in the removal of hydrocarbons and sediment from stormwater runoff before discharging into receiving waters or storm sewers.

6.5.8.3 Applicability

Oil/grit separators are typically applied to urban based drainage areas (<5ha) where ponds or wetlands are not feasible or cost effective. Separators are best applied in areas of high impervious cover where there is a potential for hydrocarbon spills and polluted sediment discharges. Typical applications include parking lots, commercial & industrial sites, petroleum service stations, airports, and residential developments (pre-treatment of ponds/wetlands or as part of a treatment train).

6.5.8.4 Effectiveness

Oil/grit separators can be effective for treatment of stormwater pollution at its source. Source control is favorable for water quality control since the dilution of pollutants in stormwater becomes problematic in terms of effective treatment as the drainage area increases. Depending on land use, drainage area, site conditions, and hydrology, some oil/grit separators may be effective in reducing TSS. See Table 6-2 for oil/grit separator design types and characteristics.

6.5.8.5 Water Quantity

Oil/grit separators implement the use of permanent pool storage for removal of stormwater pollution. However, they are not designed to provide extended detention storage, and thus provide little flow attenuation.

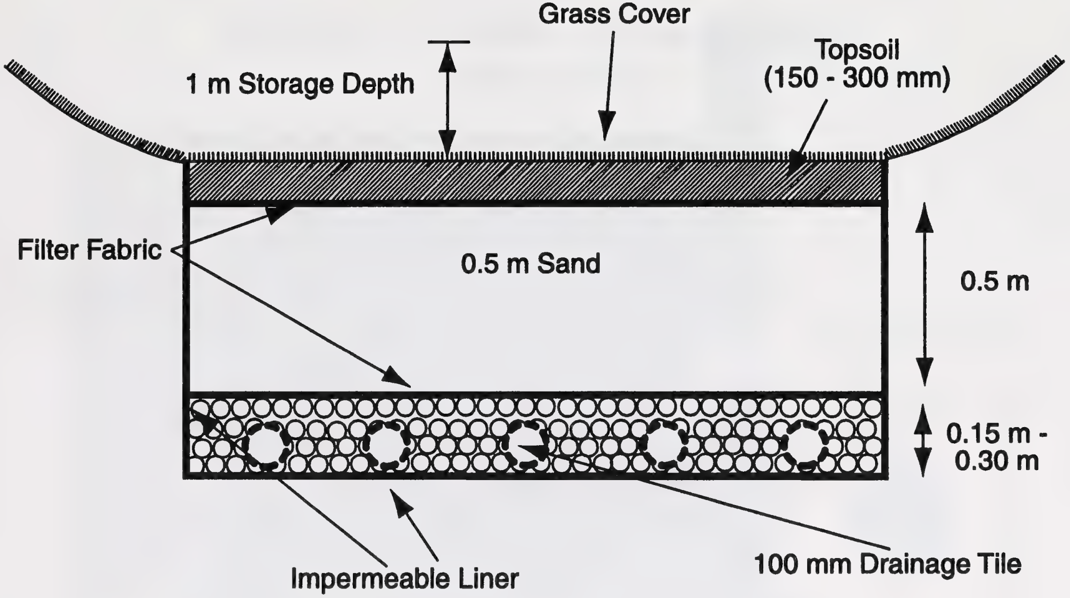


Figure 6.17
Sand Filter Cross Section Profile

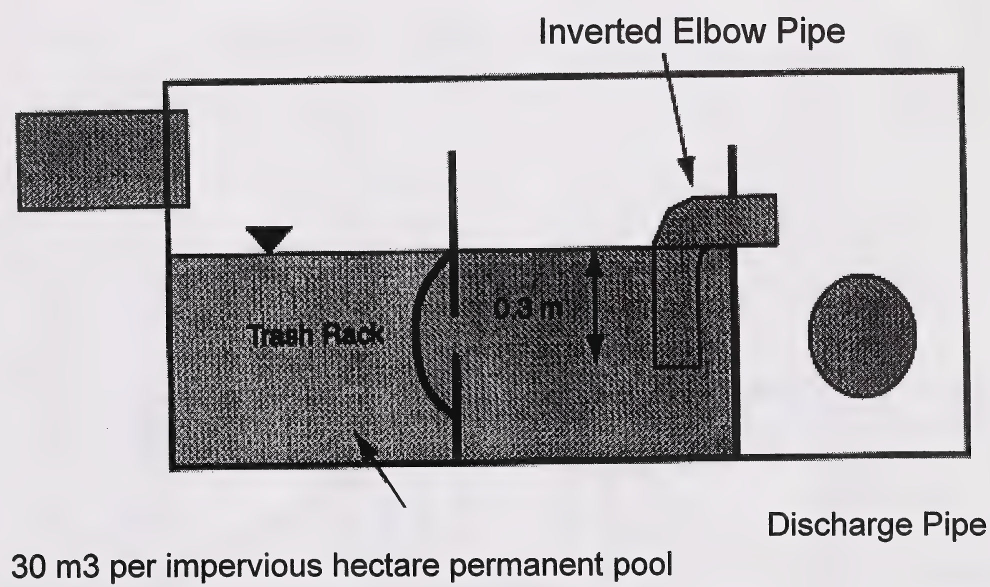


Figure 6-18
Standard 3 Chamber Oil/Grit Separator

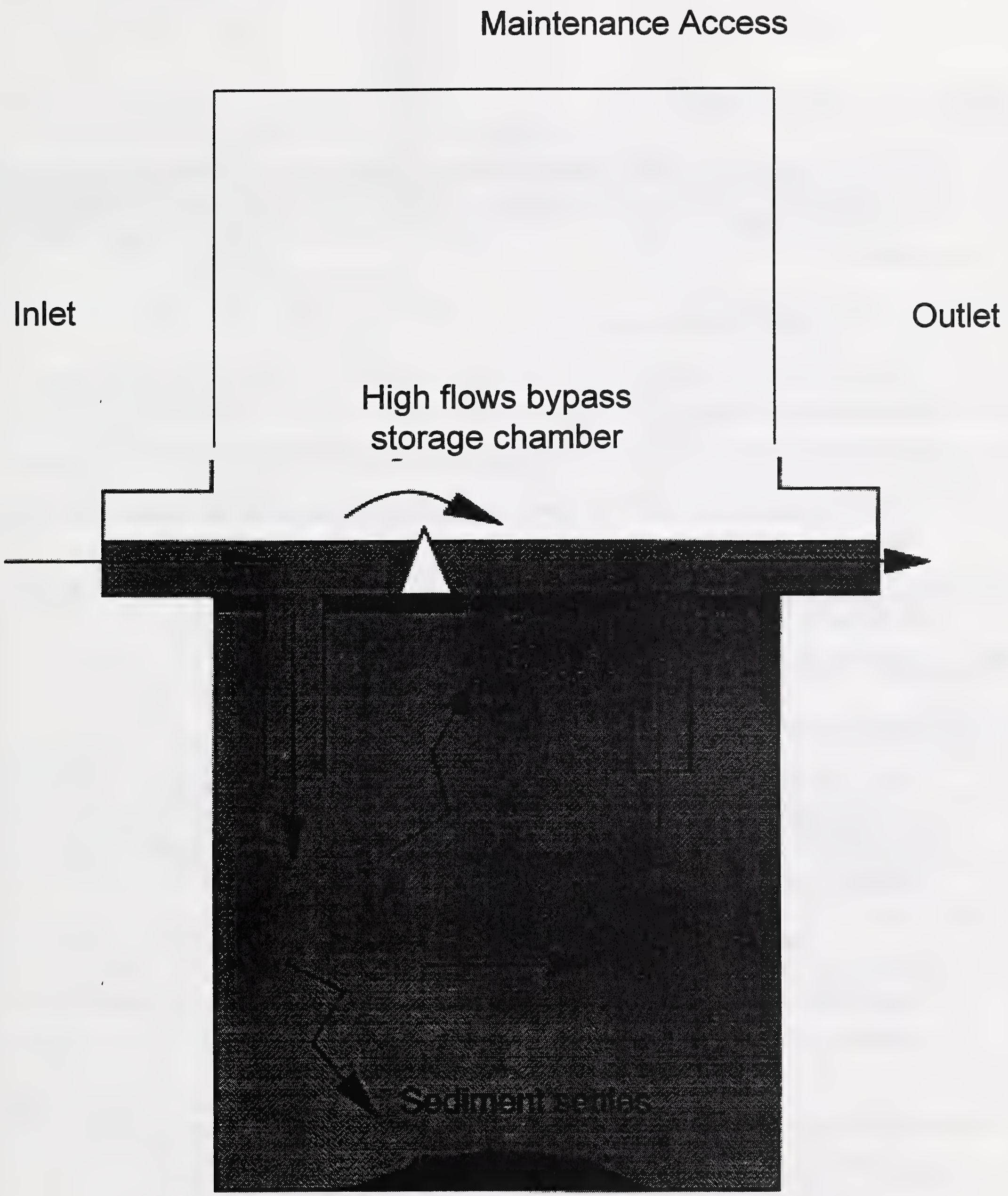


Figure 6-19
Bypass Separator

6.5.8.6 Water Quality

Oil/grit separators vary in design and performance. Separators that do not incorporate a high flow bypass have been found to be generally ineffective in removing/containing hydrocarbon and sediment pollutants, because of a continuous process of resuspension and settling of solids.

6.5.8.7 Design Considerations

Three chambered oil/grit separators operate most effectively when constructed offline. A flow splitter should be used to direct excess flow back to the conveyance system or to some other control practice. Only low flows should be directed to the separator.

Bypass separators are installed online, and high flows do not affect the performance of the unit.

See Figures 6-18 and 6-19 for illustrations of the oil/grit separators.

6.6 BMP Screening and Selection

6.6.1 Initial Screening

There are a range of stormwater BMP options available for most applications. The selection of an appropriate BMP or group of BMPs depends first on the objectives for stormwater management defined for a particular catchment area, as well as the constraints placed on the feasibility of particular BMPs by physical site factors.

Once the objectives for stormwater management are well defined and the site constraints are understood individual BMPs can be evaluated in terms of their overall effectiveness as stormwater control facilities. The evaluation of overall effectiveness must include both water quantity and water quality objectives.

Each stormwater management BMP has associated with it certain advantages and disadvantages that may reduce the viable options for stormwater management for a particular development area.

Table 6-1 summarizes the advantages and disadvantages of a number of BMPs.

Table 6-1 BMP Advantages and Disadvantages		
BMP	**Advantages**	**Disadvantages**
Wet pond	• Capable of removing soluble as well as solid pollutants • Provides erosion control • Habitat, aesthetic, and recreation opportunities provided • Relatively less frequent maintenance schedule	• More costly than dry ponds • Permanent pool storage requires larger land area • Could have negative downstream temperature impacts • Could be constrained by topography or land designations • Sediment removal relatively costly when required
Dry pond	• Batch mode has comparable effectiveness to wet ponds • Not constrained by land area required by wet ponds • Can provide recreational benefits	• Potential re-suspension of contaminants • More expensive O&M costs than wet ponds (batch mode)
Wetlands	• Pollutant-removal capability similar to wet ponds • Offers enhanced nutrient-removal capability • Potential ancillary benefits, including aviary, terrestrial, and aquatic habitat	• Requires more land area than wet ponds • Could have negative downstream temperature impacts • Could be constrained by topography or land designations • Potential for some nuisance problems
Infiltration trenches	• Potentially effective in promoting recharge and maintaining low flows in small areas • May be appropriate as secondary facility where maintenance of groundwater recharge is a concern • No thermal impact • No public safety concern	• Appropriate only to small drainage areas (<2 ha) and residential land uses • Constrained by native soil permeabilities • Usually requires pretreatment device • Potential contamination of groundwater must be investigated • Generally ineffective for water quantity control • High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance
Infiltration basins	• Potentially effective in promoting recharge and maintaining low flows in small areas • May be appropriate as secondary facility where maintenance of groundwater recharge is a concern • No thermal impact • No public safety concern	• Appropriate only to relatively small drainage areas (<5 ha) and residential land uses • Constrained by native soil permeabilities • Pretreatment is recommended • Potential contamination of groundwater must be investigated • Generally ineffective for water quantity control • High rate of failure due to improper siting and design, pollutant loading, and lack of maintenance

Table 6-1 BMP Advantages and Disadvantages		
BMP	**Advantages**	**Disadvantages**
Filter strips	• Water quality benefits may be realized if part of overall SUM plan (i.e., as secondary facility) • Effective in filtering out suspended solids and intercepting precipitation • May reduce runoff by reducing overland flow velocities, increasing time of concentration, and increasing infiltration • Can create wildlife habitat • No thermal impact	• Limited to small drainage areas (<2 ha) with little topographic relief • Uniform sheet flow through vegetation difficult to maintain • Effectiveness in freeze/thaw conditions questionable
Sand filters	• Generally effective in removing pollutants, are resistant to clogging and are easier/less expensive to retrofit compared to infiltration trenches	• Not suitable for water quantity control • Generally applicable to only small drainage areas (<5 ha) • Do not generally recharge groundwater system • May cause aesthetic/odour problems • O&M costs generally higher than other end-of-pipe facilities
Oil/grit separators (3-Chamber Separator)	• Offline, 3-chamber (oil, grit, discharge) separators may be appropriate for commercial, industrial, large parking, or transportation-related areas less than 2 ha	• Scour and resuspension of trapped pollutants in heavy rainfall events • Difficult to maintain • Relatively high O&M costs • Online design of 3-chamber separators has resulted in poor pollutant removal performance
Oil/Grit Separators (Bypass Separator)	• Bypass prevents the scouring and resuspension of trapped pollutants in heavy rainfall events • Effective in removing sediment load when properly applied as a source control for small areas • Effective in trapping oil/grease from run off	• Relatively high capital costs compared to manholes • Applicable for drainage areas less than 5 ha

6.6.2 Physical Constraints

Site characteristics may be the factor that will ultimately determine the applicability of individual or combinations of BMPs. Physical factors that need to be assessed in evaluating the suitability of BMPs include:

- Topography
- Soils stratification
- Depth to bedrock
- Depth to seasonably high water table
- Drainage area

As stated in Section 4.2, it should be recognized that development along the eastern slopes of the Rockies can be faced with totally different runoff characteristics than communities further east. The resulting runoff, both within the urban catchment and from the rest of the watershed creates some challenges in selecting BMPs to address the stormwater management concerns. For example:

- detention ponds may not be able to be used if the potential site is in a steep part of the development.

- debris dams may be required to collect and control sediment, large and small, from the upper part of the watershed.

- proper design of the overland flow channels is paramount, especially at road crossing. This includes the channel alignment and the design of erosion protection, so that damage to adjacent property does not occur.

- crossings in channels should be sized so that they convey any debris that might be envisaged.

- turbulent flow and air-entrainment may occur in channels and streams.

- periodic weirs or dams may be required in the stream or channel to dissipate energy from the fast flowing runoff.

- more conservative design of riprap in streams and at culverts.

Table 6-2 summarizes physical constraints associated with various BMP types.

Table 6-2
Physical BMP Constraints

BMP	Criteria				
	Topograph y	Soils	Bedrock	Groundwater	Area
On-Lot BMP					
Flat lot grading	<5%	none	none	none	none
Soak-away pit	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Rear yard infiltration	<2%	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<0.5 ha
Conveyance BMP					
Grassed swales	<5%	none	none	none	none
Perforated pipes	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	none
Pervious catchasins	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	none
End-of-Pipe BMP					
Wet pond	none	none	none	none	>5 ha
Dry pond	none	none	none	none	>5 ha
Wetland	none	none	none	none	>5 ha
Infiltration basin	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<5 ha
Infiltration trench	none	loam (min. infiltration rate ≥15 mm/h)	>1 m below bottom	>1 m below bottom	<2 ha
Filter strips	<10%	none	none	>0.5 m below bottom	<2 ha
Sand filters	none	none	none	>0.5 m below bottom	<5 ha

Table 6-2 Physical BMP Constraints					
	Criteria				
BMP	Topograph y	Soils	Bedrock	Groundwater	Area
Oil/grit separators (3 chamber)	none	none	none	none	<1 ha
Oil/grit separators (Bypass)	none	none	none	none	<5 ha
From MOEE, 1994 (Except Bypass Separator)					

6.6.3 Final Screening

In the initial screening phase the options for BMPs were limited by particular disadvantages and site constraints. The list of BMP options that are still considered feasible are further screened by the application of specific objectives that must be met as part of the development including:

- Water quality
- Flooding
- Erosion
- Recharge

The performance of the BMPs in regard to the objectives for stormwater management are shown in Table 6-3.

Table 6-3
Potential BMP Opportunities

Stormwater BMP	Water Quality	Flooding	Erosion	Recharge
Lot Level BMPs				
Lot grading	♦	♦	♦	●
Roof leader ponding	♦	♦	♦	●
Roof leader soak-away pits	♦	♦	♦	●
Conveyance BMPs				
Pervious pipes	●*	♦	♦	●
Pervious catchbasins	●*	♦	♦	●
Grassed swales	●	♦	●	♦
End-of-Pipe BMPs				
Wet pond	●	●	●	○
Dry pond	♦	○	●	○
Dry pond with forebay	●	●	●	○
Wetland	●	●	●	○
Sand filter	●	♦	♦	○
Infiltration trench	♦*	♦	♦	●
Infiltration basin	♦*	♦	♦	●
Vegetated filter strip	●	○	♦	♦
Buffer strip	♦	○	♦	♦
Special purpose BMP				
Oil/grit separator (3 chamber)	♦	○	○	○
Oil/grit separator (Bypass)	●	○	○	○

● Highly effective (primary control)
♦ Limited effectiveness (secondary control)
○ Not effective
* May have adverse effects
From MOEE, 1994 (Except Bypass Separators)

6.6.4 Water Quality Control and Enhancement Opportunities

In many areas of development, stormwater management practices must meet stringent water quality objectives to protect sensitive receiving waters. Water quality objectives can be defined for a stormwater management system and then appropriate BMPs can be selected from the prescreened list that will meet the water quality objectives.

The reported effectiveness, to remove pollutants, of a number of BMPs are shown in Table 6-4. The average, reported range, and probable range of the BMPs are also shown in the table. It can be seen that the efficiencies of BMPs are widely variable. They can be affected by the configuration and shape of the design, rainfall intensity, volume and duration, detention time, flow rates, runoff characteristics, and can be site specific. Solids and metals settled at the bottom of a detention pond could be re-suspended by high flows in spring next year thereby reducing its control efficiency. The size of the permanent pool in a wet pond can affect significantly the removal efficiency of total suspended solids and associated particulates. A larger permanent pool will increase the removal efficiencies. In most cases, a detention time of 12 to 24 hours will remove a large proportion of suspended solids.

Since efficiencies are affected by a large number of hydrologic and hydraulic factors, long term monitoring is necessary to measure the performance of a particular BMP under a variety of conditions. Several years of monitoring of a number of facilities are necessary to establish the average and reported range of efficiency of a particular BMP. Because of long term commitment and the high cost of monitoring, there has been little effort spent in this field. Table 6.4 provides a reference for the approximate range of efficiencies but best professional judgement is necessary in selecting the right type of BMPs for controlling the intended pollutants. A discussion on the monitoring of stormwater BMPs is given in Section 7.

Table 6-4
Effectiveness of Best Management Practices for Control of Runoff from Newly Developed Areas

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
Infiltration Basin	Average:	75	65	80	65	65	65	Soil percolation rates Basin surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1987; Griffin et al, 1980; EPA, 1983; Woodword-Clyde, 1986
	Reported Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-80	50-80	50-80	50-80	50-80	50-80		
	No. of Values Considered:	7	7	7	4	4	4		
Infiltration Trench	Average:	75	60	55	65	65	65	Soil Percolation rates Trench surface area Storage volume	NVPDC, 1979; EPA, 1977; Schueler, 1987; Griffin et al, 1980; EPA, 1983; Woodword-Clyde, 1986; Kuo et al 1988; Lugbill, 1990
	Reported Range:	45-100	40-100	(110)-100	45-100	45-100	45-100		
	Probable Range:								
	SCS Soil Group A	60-100	60-100	60-100	60-100	60-100	60-100		
	SCS Soil Group B	50-90	50-90	50-90	50-90	50-90	50-90		
	No. of Values Considered:	9	9	9	4	4	4		
Vegetated Filter Strip	Average:	65	40	40	40	45	60	Runoff volume Slope Soil infiltration rates Vegetative cover Buffer length	IEP, 1991 Casman, 1990 Glick et al, 1991 VADC, 1987 Minnesota PCA, 1989 Scheuler, 1987 Hartigan et al 1989
	Reported Range:	20-80	0-95	0-70	0-60	20-90	30-90		
	Probable Range:	40-90	30-80	20-60	-	30-80	20-50		
	No. of Values Considered:	7	4	3	2	3	3		
Grass Sawle	Average:	60	20	10	25	70	60	Runoff volume Slope Soil infiltration rates Vegetative cover Swale length Swale geometry	Yousel et al, 1965 Dupuls, 1985 Washington State, 1988 Schueler, 1987 British Columbia Res. Corp, 1991 EPA, 1983 Whelen et al, 1988 Pitt, 1986 Caeman, 1990
	Reported Range:	0-100	0-100	0-40	25	3-100	50-80		
	Probable Range:	20-40	20-40	10-30	-	10-20	10-20		

Table 6-4
Effectiveness of Best Management Practices for Control of Runoff from Newly Developed Areas

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
	No. of Values Considered	10	8	4	1	10	7		
Porous Pavement	Average:	35	5	20	5	15	5	Maintenance Sedimentation storage volume	Schueler, 1987
	Reported Range:	0-95	5-10	5-55	5-10	10-25	5-10		
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
Concrete Grid Pavement	Average:	90	90	90	90	90	90	Percolation rates	Day, 1981 Smith et al, 1981 Schueler, 1987
	Reported Range:	65-100	65-100	65-100	65-100	65-100	65-100		
	Probable Range:	60-90	60-90	60-90	60-90	60-90	60-90		
	No. of Values Considered:	2	2	2	2	2	2		
Sand Filter/Filtration Basin	Average:	80	50	35	55	60	65	Treatment volume Filtration media	City of Austin, 1988 Environmental and Conservation Service Department, 1990
	Reported Range:	60-95	0-90	20-40	45-70	30-90	50-80		
	Probable Range:	60-90	0-80	20-40	40-70	40-80	40-80		
	No. of Values Considered:	10	6	7	3	5	5		
Water Quality Inlet	Average:	35	5	20	5	15	5	Maintenance Sedimentation storage volume	Pitt, 1965 Field, 1965 Schueler, 1987
	Reported Values:	0-95	5-10	5-55	5-10	10-25	5-10		
	Probable Values:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	3	1	2	1	2	1		
Water Quality Inlet with Sand Filter	Average:	80	NA	35	55	80	65	Sedimentation storage volume Depth of media	Shaver, 1991
	Reported Range:	75-85	NA	30-45	45-70	70-90	50-80		
	Probable Range:	70-90	_	30-40	40-70	70-90	50-80		

Table 6-4
Effectiveness of Best Management Practices for Control of Runoff from Newly Developed Areas

Management Practice		Removal Efficiency (%)						Factors	References
		TSS	TP	TN	COD	Pb	Zn		
	No. of Values Considered:	1	0	1	1	1	1		
Oil/Grit Separator (3 chamber)	Average:	15	5	5	5	15	5	Sedimentation storage volume Outlet configurations	Pitt, 1985 Schueler, 1987
	Reported Range:	0-25	5-10	5-10	5-10	10-25	5-10		
	Probable Range:	10-25	5-10	5-10	5-10	10-25	5-10		
	No. of Values Considered:	2	1	1	1	1	1		
Oil/Grit Separator (Bypass)	Average:	55	25	Na	20	35	25	Storage Volume Land Use Rainfall Characteristics	Greb et al, 1998-12-14 Labatiuk et al, 1997 EST. 1998
	Reported Range:	30-90	0-100	Na	10-25	20-50	15-40		
	Probable Average:	40-80	15-35	Na	10-25	20-50	15-40		
	No. of Values Considered:	52	41	Na	15	17	17		
Extended-Detention Dry Pond	Average:	45	25	30	20	50	20	Storage volume Detention time Pond shape	MWCOG, 1983 City of Austin, 1990 Schueler and Heinrich, 1988 Pope and Hess, 1989 OWML, 1987 Wollnold and Stack, 1990
	Reported Range:	5-90	10-55	20-60	0-40	25-65	(-40)-65		
	Probable Range:	70-90	10-60	20-60	30-40	20-60	40-60		
	No. of Values Considered:	6	6	4	5	4	5		
Wet Pond	Average:	60	45	35	40	75	80	Pond volume Pond shape	Wotzka and Oberta, 1988 Yousel et al, 1986 Cullum, 1985 Driscoll, 1983 Driscoll, 1986 MWCOG, 1983 OWML, 1983 Yu and Benemouflok, 1988 Hother, 1989 Martin, 1988 Dowman et al, 1989 OWML, 1982 City of Austin, 1990
	Reported Range:	(-30)-91	10-85	5-85	5-90	10-85	10-95		
	Probable Range:	50-90	20-90	10-90	10-90	10-95	20-95		
	No. of Values Considered:	18	18	9	7	13	13		

Table 6-4
Effectiveness of Best Management Practices for Control of Runoff from Newly Developed Areas

Management Practice		Removal Efficiency (%) TSS	TP	TN	COD	Pb	Zn	Factors	References
Extended-Detention Wet Pond	Average:	80	65	55	NA	40	20	Pond volume	Ontario Ministry of the Environment, 1991 cited in Schueler et al 1992
	Reported Range:	50-100	50-60	55	NA	40	20	Pond shape	
	Probable Range:	50-95	50-90	10-90	10-90	10-95	20-95	Detention time	
	No. of Values Considered:	3	3	1	0	1	1		
Constructed Stormwater Wetlands	Average:	65	25	20	50	65	35	Storage volume	Harper et al, 1988 Brown, 1985
	Reported Range:	(-20)-100	(-120)-100	(-15)-40	20-80	30-95	(-30)-60	Detention time	Wotzka and Oberta, 1988
	Probable Range:	50-90	(-5)-80	0-40	-	30-95	-	Pool shape	Hickock et al, 1977 Burten, 1987
	No. of Values Considered:	23	24	6	2	10	8	Wetlands biota Seasonal variation	Martin, 1988 Morris et al, 1981 Sherberger and Davis, 1982 ABAG, 1979 Oberts et al, 1989 Rushton and Dye, 1990 Hay and Barrett, 1991 Martin and Smool, 1988 Ralnelt et al, 1990 cited in Woodward and Clyde, 1991

NA Not available
a Design criteria: storage volume equals 80% avg. runoff volume, which completely drains in 72 hours; maximum depth = 6 ft.; minimum depth = 2 ft.
b Design criteria: storage volume equals 90% avg. runoff volume, which completely drains in 72 hours; maximum depth = 5 ft.; minimum depth = 3 ft.; storage volume = 40% excavated trench volume
c Design criteria: flow depth < 0.3 ft.; travel time > 5 min.
d Design criteria: Low slope and adequate length
e Design criteria: minimum extended detention time 12 hours
f Design criteria: minimum area of wetland equal 1% of drainage area
g No information was available on the effectiveness of removing oil and grease
h Also reported as 90% TSS removed
i Also reported as 50% TSS removed

7.0 Operation and Maintenance Considerations

7.1 Introduction

There are a number of operation and maintenance activities that must be carried out if an urban stormwater management system is to serve the public adequately. As stormwater systems are not for primary public health protection (unlike water supply and sewage collection systems), operational considerations may not currently be a major component of a municipality's annual operation and maintenance budget. The design of stormwater management facilities has historically emphasized the need for conveyance and flood control. The introduction of new water quality design criteria for urban stormwater management facilities and BMP's that emphasize water quality necessitate changes in operation and maintenance practices. Poor practice in this area will eventually result in great expense and inconvenience to the public and can also be a factor in causing pollution problems in receiving water bodies.

Urban stormwater management facilities designed for pollutant removal present a new set of operation and maintenance issues. A number of BMP's for conveyance and stormwater quality control are not currently effective within most municipalities, partly because of operation and maintenance problems that are associated with the BMP or at least are perceived to be associated with the BMP. The implementation of BMP's such as swales and ditches as minor system components or enhanced wet ponds as major system components can have a significant impact on the level of effort required by a municipality to maintain stormwater management facilities at an optimal level of operation. Also, changes in the requirements for operation and maintenance of stormwater management facilities may introduce a different set of liability issues than may be a concern of the municipality. These liabilities may be associated with the physical appurtenances of the stormwater management facility or the accumulation of pollutants captured from runoff.

It is important for a municipality to have in place an effective and efficient operation and maintenance strategy. Such a strategy can limit the liabilities and additional costs that may be associated with maintaining urban stormwater quality control facilities. Also, deferred maintenance of urban stormwater quality control facilities often results in failure of the systems and accumulation of pollutants above acceptable limits.

This section outlines the operation and maintenance requirements associated with urban stormwater control alternatives that are part of the current minor system conveyance and flood control practices and those additional stormwater quality control enhancements that are recommended as BMP's.

7.2 Conventional Operation and Maintenance

Most minor system components are subject to problems such as erosion, clogging, and collapse. Maintenance is required to preserve capacity within the system. Maintenance activities for the minor system can be divided into preventive and corrective measures.

7.2.1 Preventive Maintenance

Preventive maintenance comprises the inspection of the system, record keeping, regular maintenance and the analysis of data related to past complaints and problems.

Routine inspections should be carried out on all minor system components in association with regular scheduled maintenance such as catchbasin cleaning and street sweeping. The frequency of these maintenance requirements is generally sufficient to provide adequate inspection opportunities for minor system components. Inspection requirements should be documented for each type of system component and the information transferred to adequate inspection records. More detailed inspections, such as camera inspections of sewer infrastructure components, should be carried out at some frequency as determined by the municipality. The frequency should be specific to each type of system component and reflect historical maintenance problems, the age of the system, and the operational parameters such as stormwater quality and discharge rates.

Record keeping is essential to effective maintenance of the stormwater drainage system. Complete records should be kept of all storm sewer system components, including:

- When the sewer was constructed (also, the designer and contractor),
- Type, size, and shape of pipe,
- Service area and land use,
- Manholes and catchbasins (location, type, and inverts),
- Inspections (date, methods, location, and results),
- Complaints (location, nature, date, time, weather), and
- Repairs and replacements.

A record-keeping system or information management system is recommended to maintain a useful database. The physical data should be recorded on "as-constructed" drawings, which include the plan and profile of all sewers. As well, overall composite drawings of the entire system are valuable for containing much of the above information. Composite drawings with system information should be in reproducible form and be updated as changes occur. The options for information management that can be selected by the municipality range from hard-copy filing to computer-based software packages that are used to record specific inventory data like Sewer Information Management System (SIMS) to complete Geographical Information Systems (GIS) that integrate system data, records, and mapping requirements into a single package that may include all municipal information such as sanitary sewers, storm sewers, water supply, surface water resources, water quality, and roads. The GIS system is the most advanced and efficient tool but may be cost-prohibitive for some municipalities.

Regular maintenance is required to ensure the function of the stormwater drainage system. During a rainfall event, it is important to be able to provide adequate flood control to limit the potential for property damage and personal injury. Regular maintenance should include the following activities:

- Cleaning and flushing of streets,
- Sediment removal from catchbasins,
- Supervision of connections and disconnections,
- Steaming of frozen catchbasins, outfalls, and culverts,
- Inspection of pipe condition by visual or camera techniques,
- Repair or replacement of damaged pipe, manholes, catchbasins, and other appurtenances, and
- Review and updating of records.

7.2.2 Corrective Maintenance

Corrective maintenance is unscheduled, relating mainly to emergency situations. It pertains to items requiring immediate repair such as pipe breaks, collapses, or washouts. Corrective actions can be taken to reduce flood potential, to limit liability, to prevent personal injury, or to protect the environment.

7.2.3 Maintenance Responsibilities

The municipality is the authority ultimately responsible for the operation and maintenance of urban stormwater management facilities.

7.3 Operation and Maintenance of Detention Facilities

7.3.1 Wet Ponds

Wet ponds have a permanent pool of water stored at all times. Concerns with algae growth, mosquitos, and overgrown vegetation have to be addressed. A maintenance schedule should be established for the summer and winter seasons.

7.3.1.1 Maintenance Responsibilities

Regularly required activities for the aesthetic appearance and recreational utility of the facility include lawn mowing, shrub trimming, debris removal, and ice thickness monitoring. Maintenance activities related to the water body and control structures and include water quality observation, aquatic weed control, and sediment removal.

Unscheduled maintenance will be required from time to time in response to extreme rainfalls or prolonged dry periods. These activities include attendance at detention facilities when they are at flood levels, embankment and shoreline repairs, freeing outlets plugged by debris or ice, low-water-level control, and the handling of algal blooms. The frequency and cost of these maintenance activities depend on the season, type of pond (wet or dry), the size of the facility, and the objectives of the municipality for the area as part of its landscape and recreational facilities.

7.3.1.2 Equipment Access

It is imperative that access for maintenance equipment be included in the design of detention facilities. Rapid access to inlet and outlet works, although infrequently required, must be provided. These access points should not be obstructed by fences, landscaping, or other works. Access to wet ponds must provide for the eventual need for scrapers or crawler tractors and trucks to remove sediment accumulation on the pond bottom. Sediment removal will also require a means for draining the pond (either built into the outlet structure or by pumping).

7.3.1.3 Turf and Landscape

Turf and landscape maintenance can be the most significant and costly portion of a detention facility's maintenance budget. It is a regular activity during the spring, summer, and early fall.

Detention facilities require grass cutting and tree pruning in public areas to the same extent as any other park area. It is advisable to have a limited number of trees in the grassed area around a pond to facilitate operation of ride-on lawnmowers for fast and efficient cutting of grass. Trees should either be spaced far apart or planted in clusters.

In residential areas where aesthetics are a concern of land owners, grass cutting in dry ponds and around the perimeter of wet ponds, may be required during the summer months. Grass cutting will not, however, enhance water quality and should, therefore, be done as infrequently as possible. When cutting grass, care should be taken to ensure that grass clippings are not ejected into the detention pond. Grass cuttings add organic loading to the pond.

7.3.1.4 Debris Removal

Debris and litter are significant maintenance issues. Frequent maintenance visits are required to empty litter containers, pick up wind-blown litter, remove floating debris, and check for vandalism. Inlet and outlet structures should be occasionally checked for blockages. Debris should be removed from the site especially the inlet and outlet structures at each mowing and if required, after major storm events.

7.3.1.5 Outlet Valve Adjustment

The effects of detention times on water quality may vary from one pond design to another. The outlet valve should be adjustable in order to control the detention time and the resultant water quality discharged from the pond. Outlet adjustments should be based on discharge water quality criteria.

7.3.1.6 Control of Weeds, Aquatic Weeds, and Algae

Weeds around detention facilities are described as any unwanted vegetation. Weeding should be done by hand to prevent the destruction of surrounding vegetation. The use of herbicides and insecticides should be prohibited near wet ponds since they create water quality problems. The use of fertilizer should also be limited to minimize the nutrient loadings to the downstream receiving waters.

The growth of aquatic weeds in detention ponds is affected by the water depth, turbidity, and the availability of nutrients. Water depth is the major factor for the control of emergent vegetation. When the depth exceeds 1.2 m, emergent vegetation is rarely a problem. This still leaves a potential for weed growth around the perimeter of the pond. Soil sterilization with a chemical will restrict growth in this zone for 2 or more years. After this period a number of options can be pursued:

- Accept the perimeter growth. Many people do not consider emergent vegetation, such as cattails, unsightly.

- Cut and remove the weed growth from either the land or the water. This will be a short-term solution (annual removal will likely be required). The feasibility of this will depend on the condition of the pond sideslopes.

- Drain the pond, remove the weed growth, and re-sterilize the perimeter soil. If chemicals are to be used, the municipality should check with Alberta Environmental Protection to ensure that their use is permissible.

- Lower the water surface for a period of time (such as over the winter) to kill the growth, then reestablish the water level.

The selected method for weed control is a matter of choice, although tolerating the growth is the most economical and also protects water quality. The other alternatives involve environmental and aesthetic consequences that must be considered based on local attitudes.

Algal growth will occur in any water body that has an adequate supply of nutrients. These are usually available in detention ponds unless specific maintenance effort is directed at sediment and nutrient removal. Prolonged warm weather encourages algal growth. Algal blooms are most likely to occur in areas of the lake adjacent to the inlets, and are most effectively treated by chemical application; e.g. alum, lime, etc. Only pesticides approved by Alberta Environmental Protection may be used. Also, the timing of the application of any chemical products is very important.

7.3.1.7 Mosquito Control

Some jurisdictions have used chemical sprays or pellets applied to the pond surface to control mosquitoes. In Winnipeg, fish have selectively been used for mosquito control. Agitating the pond surface by circulating water or using aeration equipment has also been

used to reduce mosquito populations. Grass cutting may provide a supplementary benefit by somewhat reducing adult mosquito populations near a pond.

Mosquito problems in wet ponds are increased if the water levels in the ponds experience large fluctuations, if aquatic plants are allowed to grow, and if the water level in the ponds does not drain to normal levels within a few hours after a rainfall event. Also, pond shape should allow for adequate water circulation patterns and wind disturbance in the ponds.

7.3.1.8 Aeration and Circulation

Aeration and circulation equipment can be used to make the pond environment less conducive to the production of algae and mosquitoes. Some researchers, however, have found that aeration has enhanced algal production where oxygen, not sunlight or nutrients, is the limiting factor. Aerating the lakes generally aids the decomposition of algae and other dead biomass. This helps alleviate odour but is a cure rather than a preventive measure. A better approach is to prevent excessive growth of algae by reducing nutrient inflow by trapping sediment upstream of the pond inlets. If the growth of weeds and algae is controlled, recycling of nutrients through decomposition is also reduced. In addition, oxygen levels will not be depleted, and odour problems can be avoided.

A wet pond should be designed so that "dead bay" areas in the pond are avoided. The strategic location of inlets and outlets may generate sufficient flushing action to achieve this objective. Aeration and circulation equipment should not be mandatory in the design of detention facilities. Their value is for retrofitting to solve problems that have occurred, or to improve aesthetics.

7.3.1.9 Signage

Warning signs should be posted along the perimeter of wet detention facilities to prohibit activities that may present a danger to public health or possible interference with the operation of the facility. Additional signage is required on a seasonal basis in regard to unsafe ice conditions and the application of weed control chemicals. Signage requirements for various weed control applications must meet regulations as approved by Alberta Environmental Protection. Posting of fines for throwing rocks or other debris into the wet pond is meant to discourage such activities.

7.3.1.10 Makeup Water

Makeup water can also assist in the control of mosquitos and algae if it is introduced in a turbulent fashion at the right locations. In addition, if the makeup water has low contaminant concentrations it may assist the water quality situation in the pond and pond discharge by dilution. The most appropriate source for good quality makeup water is the municipal system. Due to the high cost of this water, it should be utilized only for water quality control and not water level control. A lake with the appropriate sideslopes and shoreline treatment will not show unsightly "mudflats" when the water falls below normal water level. In most

of Alberta, groundwater is not an appropriate source of makeup water. Generally, groundwater is a scarce resource and makeup water for a detention pond is not a high-priority use.

7.3.1.11 Winter Activities

Maintenance activities for detention facilities are reduced and are of a dramatically different nature during the winter. Wet ponds require the most maintenance attention during the winter months. The most active periods are during freeze-up in early winter and during the spring thaw.

Many stormwater detention ponds are used for ice-skating during winter. If the municipality chooses to support this recreational activity for the community, it can become involved in the establishment and maintenance of skating facilities. Otherwise, the municipal authority should be quite clear that people use the lakes at their own risk and should leave it to them to clear skating areas.

At the beginning of winter, some municipalities place announcements in the local newspapers or on radio to alert the residents that it may be unsafe to use the lake surface during winter. Others present a firmer position, and indicate by the placement of permanent "thin ice" signs around the pond that use of the lake surface is at the public's own risk (an implicit responsibility). Considering the possibility of unsafe ice conditions elsewhere on the pond's ice surface (the area not being maintained by the municipality), the latter policy has considerable merit.

The selection of skating areas is primarily determined by the accessibility of the sites to maintenance equipment and user facilities for the public. On some ponds the entire surface area may be kept free of snow to allow skating. On others, one or more local cleared areas can be used. Where isolated areas are to be used, these should be located in quiet water areas, away from any inlet or outlet works in order to minimize the possibility of localized thermal erosion of the ice. Areas that have this problem should be barricaded, fenced, or have signs to signal the danger.

Natural ice grows downward from the water/ice interface. As the ice thickness increases, the rate of growth decreases (it becomes more difficult for the heat in the warm water to escape to the cold atmosphere). On the other hand, flooding the ice surface will result in the establishment of an ice surface more quickly. This is mainly because the primary freezing surface is in direct contact with the colder atmosphere. In either case, it is important to keep the ice surface free of snow during cold weather as the insulating effects of snow greatly reduce the rate of ice growth.

An ice surface is not safe to carry a load until it has reached a sufficient thickness. Typically, holes are drilled at several locations to check the area for ice thickness and uniformity. Drilling is done frequently until it is possible to support snow clearing equipment. Once this "load test" has been completed there is no need for further flooding (other than for ice-surface restoration) or drilling as long as the weather remains cold.

In the spring, an ice surface deteriorates and usually floods with meltwater prior to the ice becoming unsafe to walk on. This condition hopefully discourages use of the lake during the winter/spring transition. As the spring melt progresses, children are sometimes tempted to walk or bicycle on the lake surface, when it is unsafe to do so. In the interests of public safety, maintenance of a skateable ice surface should cease at the earliest signs of a warming trend. Frequent site visits during thawing periods (particularly after school and on weekends) should be scheduled to discourage activities on the pond.

7.3.1.12 Sediment Control

Sediment control is one of the most important activities in maintaining the water quality of a stormwater detention facility. It is also one of the activities that is the easiest to ignore (in the short term). Stormwater detention facilities will serve as a sink for most materials that impair water quality; the facilities will improve the quality of the discharge because the detention period will allow the settling of suspended material and parameters that are bound to it. Some parameters may also be reduced by detention through biological activity, volatilization, or die-off. However, the continued discharge of sediment to the facility will ultimately degrade the pond's water quality to the point where it may be unacceptable to the public from either an aesthetic viewpoint (because of algal blooms) or a health viewpoint.

Monitoring of certain water quality parameters is required to ensure that the detention facility does not deteriorate beyond acceptable standards. Sediment control is the best means to prevent unacceptable deterioration of water quality from occurring. Although sediment is only one water quality parameter that can have adverse impacts on the environment, it is the most easily observed. As many other contaminants are associated with sediment, control of sediment will in effect control most of these substances.

In the post-development period, the first opportunity for sediment control comes at the catchbasin sumps. These sumps are used in many cities in Alberta. Catchbasins can be inconvenient to maintain as they require the servicing of a large number of minor facilities over the serviced area. Since current sewer systems are designed with self-scouring velocities, it can be more convenient to install a smaller number of large sediment traps in the system. These can be located near the inlets to the storm ponds. Sediment-removal structures located upstream of a pond can be reliably serviced on a frequent basis by eductor trucks. Servicing these facilities is required after each significant rainfall event and during the spring snowmelt runoff period.

Most stormwater systems convey sediment into the detention facility. The City of Winnipeg indicates that the sediment accumulation in its facilities is in the order of 2.5 mm per year. Winnipeg monitors the bottom topography of its lakes every 5 years to determine the need for widespread sediment removal. The original philosophy regarding sedimentation was that after a prolonged period of development, dredging or draining and excavation would have to be undertaken to restore the pond bed to the originally designed elevation. If necessary, this would occur after full urbanization of the drainage basin has occurred and sedimentation

had dropped to negligible rates. After an urban watershed has been fully developed it was expected that there would be little need to again consider dredging for many years (in the order of 20 to 50 years). Experience in the Prairies to date indicates that few ponds, if any, have required any sediment removal. The need for ongoing sediment control and removal is related to water quality control and not to preventing a loss of storage volume. In general, an unacceptable level of water quality deterioration in the wet pond discharge may be a decline in sediment removal efficiency of greater than 5 percent of the ponds original removal efficiency. This deterioration in TSS removal efficiency can be used as a guideline for calculating the frequency of sediment removal required to maintain the water quality function of the pond.

7.3.1.13 Harvesting Aquatic Plants

Many nuisance problems associated with wet ponds such as mosquitos and odour as well as more serious problems such as toxicity resulting from the growth of algae such as *Anabaena* and *Anacystis* can be controlled through the removal of aquatic growth by mechanical harvesting.

Small mechanical harvesters can be brought to the site and used to harvest aquatic plants. More common is the use of chemicals for algae and plant control. Another alternative is the use of mechanical barriers on the bottom of the ponds. This can be costly and presents problems with some methods of sediment removal. A more practical and less costly alternative may be to design the pond with a greater depth, thus reducing light penetration and aquatic growth.

7.3.1.14 Monitoring

Monitoring is necessary to evaluate whether there is a need for corrective actions to be taken to ensure that the objectives for the protection of priority resources and receiving waters are met. A monitoring program can identify the environmental conditions that indicate whether success has been achieved in meeting the stormwater control objectives. By comparing monitoring results, improvements can be incorporated into future facilities.

Under current BMP practices, there has been little monitoring due to high cost, unclear responsibilities, and the requirement for long term commitment in order to measure the performance and compliance level of the stormwater BMP facilities. Hence, monitoring has been conducted mostly as research projects funded by government agencies. Most of the facilities installed to date have focused on maintenance rather than monitoring, assuming that if they are properly maintained, their intended performance will be achieved.

It is unnecessary to monitor all stormwater BMP installations since over-monitoring will discourage the installation of BMP controls. This may not be a good resource commitment. It is recognized that there is a need for monitoring, but this is necessary only for a reasonable number of designs. To minimize cost, monitoring can be selected for unique and untested sites, highly sensitive locations, or representative sites. Data from a reasonable number of sites can be extrapolated to similar designs with occasional spot sampling using best professional judgement.

Monitoring practices for stormwater detention ponds vary widely. Of the cities who monitor water quality, the sampling frequency ranged from weekly to twice a year. A monthly monitoring program would represent a reasonable monitoring practice for the first two years after the installation of the facility. During the first two years, the installer, for example, the developer of a new subdivision, would be responsible to carry out a monitoring program to ensure that the design selected for monitoring is functioning as intended. The extent of monitoring depends on site specific requirements or constraints such as receiving water quality, sensitivity of discharge location, and impact of urbanization. When the facility is proven after two years, the maintenance and monitoring of the facility will become the responsibility of the municipality. At which time, once or twice a year of sampling is sufficient provided that the facility is properly maintained.

During the first two years after installation, in addition to monitoring the water quality in the BMP, it is important to monitor the effectiveness of the BMP in removing the various pollutants. To accomplish this, the monitoring data collected during typical operating seasons, including winter and snowmelt conditions, should be analyzed. If a BMP is not shown to be operating as required, corrective measures should be taken and effectiveness demonstrated.

Where a wet pond is intended to be a multi-use facility (i.e. allowing secondary and tertiary recreational activities), the quality of water in the pond should compare with the Surface Water Quality Objectives established by Alberta Environment Protection. Except for a short period following a runoff event, the quality of water in a pond should be expected to meet these objectives. Where concentrations exceed these objectives significantly, particularly coliform counts, the source of such contaminant loads should be found and eliminated. Typical pollutant parameters for monitoring stormwater BMP facility performance may include:

- total suspended solids (TSS)
- biochemical oxygen demand (BOD)
- dissolved oxygen (DO)
- bacteria
- toxic chemicals (such as lead, zinc, copper, mercury, etc.)
- nutrients (such as total phosphorus, total kjedahl nitrogen, nitrite, nitrate, etc.)

As it is inevitable that the public will have some contact with the water, monitoring of bacteria is important to determine the risk to public health. Control of BOD and dissolved oxygen levels ensures that the water body remains aerobic and lessens the likelihood of odour problems. Turbidity, colour and odour all relate to aesthetic considerations which may or may not be a problem depending on the user's needs and the location of the BMP. Monitoring nutrients can aid in predicting nuisance problems associated with algal blooms, which also present aesthetic concerns. The frequency of monitoring of toxic chemicals should be determined on a site specific basis. For example, quarterly monitoring is appropriate for ponds.

Stocking a pond with fish may be desirable from a recreational perspective, it is not desirable from a public health perspective. The contaminants (chemical and biological) present in stormwater are likely to accumulate in the fish flesh and can be transmitted to humans. Where stormwater ponds are stocked, both fishing and the consumption of fish should be forbidden, and fish monitored quarterly for toxic chemicals each year.

Year-round monitoring of the hydraulic properties of infiltration BMP's will provide an opportunity to monitor the quality of exfiltrated water, and hence the potential for groundwater contamination. Monthly measurements of groundwater quality may be carried out to provide sufficient detail on the effect of infiltration BMP's on groundwater quality. Continuous monitoring of percolation can be conducted to determine the effectiveness of infiltration BMP's under freeze/thaw conditions in the soil. In this case, continuous monitoring may be necessary so that the BMP effectiveness during the spring freshet, and the variation in effectiveness with season, can be determined.

Continuous water quantity and quality monitoring for surface storage BMP's in the first two years of installation could be carried out at the inlets and outlets of the BMP facility. In most cases, this will result in two monitoring locations.

7.3.1.15 Inspections

Inspection programs of wet ponds should include, at a minimum, the following points:

- Is the pond level higher than the normal permanent pool elevation more than 24 hours after a storm? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.)

- Is the pond level lower than the normal permanent pool elevation? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris of blockage.)

- Is the vegetation around the pond dead? Is the pond all open water (no bulrushes or vegetation in the water)? Are there areas around the pond with easy access to open water? (This will indicate a need to revegetate the pond.)

- Is there an oily sheen on the water near the inlet or outlet? Is the water frothy? Is there an unusual colouring to the water? (This will indicate the occurrence of an oil or industrial spill and the need for cleanup.)

- Check the sediment depth in pond. (This will indicate the need for sediment removal. The sediment depth can be checked using a graduated pole with a flat plate attached to the bottom. A marker (pole, buoy) should be placed in the pond to indicate the spot(s) where a measurement should be made. A visual inspection on the pond depth can also be made if the pond is shallow and a gradated marker is located in the pond.)

7.3.2 Wetlands

A proper mixture of plants must be grown to maintain the efficiency and the functions that wetlands provide. The lifecycle of plants and vegetation, sediment buildup, and impact of seasonal climate and weather would dictate the schedule of maintenance.

7.3.2.1 Water Level Controls

The maintenance of wetlands is carried out, to a large degree, by the control of water levels. The growth of plant and tree species that are unwanted in the wetland can be accomplished by prolonged flooding of the wetland. Species such as cattails can be reduced by prolonged flooding. Trees, shrubs, emergents, and aquatic plants can be controlled by the periodicity and length of the flooding.

7.3.2.2 Sediments

The rate of accumulation of sediments in a wetland is a function of the land use within the watershed tributary to the wetland. Areas undergoing development may subject a wetland to extremes in sediment loading if construction-site controls are not adhered to. Established urban areas may contribute small amounts of sediment more through aerial deposition and road runoff. Street-cleaning practices in the watershed can affect the amount of sediment load to a watershed.

Design of an urban stormwater wetland may incorporate an upstream catchbasin to collect sediments prior to discharge to the wetland. Maintenance of catchbasins should include removal of these sediments on a periodic basis. The frequency of sediment removal is dependent on the nature of the upstream development and generation of sediments.

7.3.2.3 Harvesting Organic Material

Nutrient loadings to an urban wetland may promote the growth of excessive vegetation. The accumulation of organic material from this growth in the wetland can limit the assimilative capacity of the wetland. Harvesting of the leaves and stems of plants may limit the buildup of organic material. The frequency of harvesting depends on the nutrient loading and the rate of plant growth.

7.3.3 Dry Ponds

Dry ponds are often used for dual purposes. If properly designed, dry ponds can be used as recreational amenities such as baseball fields in the summer and skating rinks in the winter. Maintenance of a dry pond for its stormwater control function is very minimal and usually involves grass cutting, debris removal, and inspections. Sediment control may also be an issue depending on the design of the facility.

7.3.3.1 Grass Cutting

Grass cutting should be done, at a minimum, twice per year. The frequency of grass cutting will depend on the municipal requirements for weed control and the designed dual usage, if any, of the dry pond.

For some dry ponds, aesthetics have not been a high priority during their design. Grass cutting has frequently been ignored for these facilities either intentionally or due to wet conditions caused by poor drainage across the pond bed. This type of pond should not be located near residential areas as it can become an eyesore. Fencing to preclude access should be avoided (safety should be incorporated into the design even if aesthetics are not).

7.3.3.2 Weed control

Weed control is carried out to meet municipal requirements. This can be accomplished through grass cutting or the application of chemicals. Chemical usage may affect downstream water quality. Posting of chemical usage must be carried out to meet the requirements of the municipality and Alberta Environmental Protection.

7.3.3.3 Debris Removal and Vandalism

Debris removal and inspection of the inlet and outlet controls should be carried out at least twice a year, in the spring and fall. Debris removal can also be carried out during grass cutting and regular inspections. If the dry pond is used for a dual recreation purpose it can be expected that some maintenance may be required as a result of vandalism. Vandalism can be a problem involving signs, landscaping, and the grating on the inlet pipe.

7.3.3.4 Signage

Signs posting the proper use of the dry pond as a recreational facility should be posted. Fines regarding the improper use of the dry pond should also be posted to reduce any activities that may present a danger to public safety or reduce the effectiveness of the dry pond as an urban stormwater control facility.

7.3.3.5 Inspections

- Is there standing water in the pond more than 24 hours after a storm? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.)

- Is the pond always dry, or relatively dry within 24 hours of a storm? (This could indicate a blockage of the inlet or a water quality/erosion control outlet that's too large. Visually inspect the inlet structure for debris or blockage.)

- Is the vegetation around the pond dead? Are there areas around the pond with easy access to open water? (This will indicate a need to revegetate the pond.)

- Is there a visible accumulation of sediment in the bottom of the pond or around the high waterline of the pond? (This will indicate the need for sediment removal.)

7.4 Operation and Maintenance of Infiltration Facilities

Developed urban areas often have less groundwater recharge because of increases in impermeable ground cover. The implementation of infiltration facilities for the control of stormwater can increase the recharge potential in a catchment area, decrease overland runoff during storm events, and maintain baseflows in adjacent streams during dry weather periods. Potential groundwater contamination must be a consideration when recharging groundwater resources through stormwater infiltration facilities.

7.4.1 Infiltration Basins

A common problem with infiltration basins is the buildup of sediments and debris during a long dry weather period and after rainfall events.

7.4.1.1 Sedimentation

Excess sedimentation in infiltration basins will reduce the infiltration potential of the basin. Particular problems with large amounts of sediment and oils from road surfaces and parking lots have been recognized. Maintenance of the infiltration basin must include sediment removal. Vacuum trucks can be used to remove the accumulated sediments without damaging the bottom of the basin. Drop inlets or sedimentation boxes can effectively remove sediment prior to the basin and reduce maintenance.

Many options are available for maintaining the infiltration potential of the basin. These methods include tilling, which has been shown to have very little potential. Diffusion wells have been shown to increase the infiltration potential of the basin. It is suggested that the planting of deep-rooted legumes in an infiltration basin may be beneficial in maintaining the porosity, and hence, infiltration potential in a soil.

7.4.1.2 Inspections

- Is there standing water in the basin more than 24 hours after a storm? (This will indicate a decrease in the permeability of the underlying soils and, depending on the depth of water in the pond after 24 hours, the need for maintenance: sediment removal and roto-tilling of soils. If there is greater than one-third the design depth of water in the pond 48 hours after a storm, the basin needs to be maintained.)

- Is the pond always dry, or relatively dry within 24 hours of a storm? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)

- Is there a visible accumulation of sediment in the bottom of the pond or around the high waterline of the pond? (This will indicate the need for sediment removal.)

7.4.2 Infiltration Trench

Infiltration trenches are normally constructed with a sand filter layer covered with a layer of gravel. Filter cloths may also be used to protect the gravel or stone from clogging. Pretreatment is provided, if required by sedimentation inlet controls.

Most maintenance efforts for infiltration trenches involve removal of sediment from inlet control devices. If sediment controls are not provided the infiltration potential of the trench will be significantly reduced. Maintenance of the infiltration trench at this point usually involves reconstruction of the trench.

7.4.2.1 Inspections

- Is the trench draining? Inspect the depth of water in the observation well. If the trench has not drained in 24 hours, the inlet and pretreatment SWMPs should be cleaned (that is, oil/grit separator, catchbasins, or grassed swales). If the trench has not drained within 48 hours it may need to be partially or wholly reconstructed to maintain its performance.

- Is the trench always dry, or relatively dry within 24 hours of a storm? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)

7.4.3 Filter Strips

Filter strips are vegetated areas over which diffused stormwater flows are directed by a level spreader. Filter strips are usually designed for very small areas of runoff. The objective of the filter strip is to slow down the surface flow, allowing infiltration and sediment removal. Sediment can be removed from the upstream end of the filter strip using conventional small grading equipment such as Bobcats. Maintenance activities for filter strips also involve maintaining the vegetated cover.

7.4.3.1 Inspection

The following is a list of items that should be inspected:

- Are there areas of dead or no vegetation downstream of the level spreader? (This will indicate the need to revegetate the filter strip.)

- Are there indications of rill erosion downstream of the level spreader? (This will indicate the need to revegetate the filter strip. The rill erosion may be caused by a nonuniform spreader height. The spreader should be checked near the erosion areas to determine if it is in need of repair.)

- Is there erosion of the level spreader? (The spreader should be reconstructed in areas where the spreader height is non uniform.)

- Is there standing water upstream of the level spreader? (This will indicate that the level spreader is blocked. The level spreader should be checked for trash, debris, or sedimentation. The blockage should be removed and the spreader reconstructed, if necessary.)

7.4.4 Buffer Strips

Buffer strips are vegetated areas placed adjacent to a receiving water body. They are sometimes heavily vegetated. No direct engineered controls are usually used to protect the buffer strip from sediment loads. Sediments normally accumulate in the buffer strip and vegetation is allowed to grow unchecked through the buildup of soil.

Maintenance of the buffer strip includes revegetation of any areas where vegetation fails to establish itself.

7.4.4.1 Inspection

A scheduled field inspection should be conducted to determine if there are areas of dead vegetation along the buffer strip. This will indicate the need to revegetate the buffer strip.

7.4.5 Sand Filters

Sand filters are constructed as surface filters or subsurface end-of-pipe systems. Surface sand filters can be vegetated. Maintenance of surface sand filters that do not have a vegetative cover includes periodic raking to reduce surface compaction and clogging to increase infiltration potential. Raking also removes debris from the filter. If pretreatment of sediments is provided though an inlet sedimentation device, sediment removal will be required to maintain the inlet control.

7.4.5.1 Inspections

The items requiring field inspections are listed as follows:

- Are there areas of dead vegetation in a grass-surfaced sand filter? (This will indicate the need to revegetate the filter surface.)

- Is there standing water in the filter more than 24 hours after a storm? (This will indicate a blockage in the filter, possibly in the perforated-pipe collection system or sedimentation on the surface or in the sand layer. The outlet collection system should be inspected for blockage. If there is water in the filter 48 hours after a storm, sediment removal should be undertaken. If sediment removal does not improve the performance (drainage) of the filter, the filter may need to be reconstructed.)

- Is the filter always dry? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)

- Is there a visible accumulation of sediment in a grass-surfaced sand filter? (This will indicate the need for sediment removal.)

7.4.6 Oil/Grit Separators

- **(3-Chamber Separator)** – Multiple chamber or tank-sized separators are difficult to maintain and are, therefore, prohibitive from a maintenance and operation standpoint. Manual cleaning with shovels is often required. Cleaning frequencies are higher (three to four times per year and after any known spills) and add to the maintenance difficulties and high operation costs.

- **(Bypass Separator)** – Bypass separators are easily maintained by vacuum truck. No entry into the unit is required for maintenance. Cleaning of the Bypass Separator is usually carried out once per year or after any known spills have occurred.

7.4.6.1 Inspection

- **(3-Chamber Separator)** – Sediment accumulation can be measured using a gradated pole with a flat plate attached to the bottom. The pole should be gradated such that the true bottom of the separator compared to the cover/grate is marked for comparison. Oil accumulation may be inspected from the surface for trash/debris and/or the presence of an oil/industrial spill. An oily sheen, or frothing or unusual colouring to the water will indicate the occurrence of an oil or industrial spill. The separator should be cleaned in the event of spill contamination.

- **(Bypass Separator)** – Sediment accumulation can be easily measured from the surface by removing the maintenance cover. Sediment depth can be measured from the surface without entry into the separator via a dipstick tube equipped with a ball valve (Sludge Judge). Similarly, the presence of oil can be determined by inserting a dipstick tube into the separator. Maintenance of the separator should be performed once sediment and oil accumulations exceed the manufacturers guidelines.

7.4.7 Soak-away Pits

Soak-away pits are normally used only for infiltration of relatively clean stormwater from areas such as rooftops. This reduces clogging and maintenance requirements.

Soak-away pits can generally be left without any maintenance unless significant overflows occur during average rainfall events. Otherwise the filter should be cleaned once a year, preferably after the leaves have fallen off the trees in the late fall.

7.4.7.1 Inspection

The soak-away pits should be inspected to determine if there are frequent overflows to the surface during small storm events. Frequent overflows will indicate that the roofleader filter has clogged or the soak-away storage media has become clogged. The filter should be checked for an accumulation of leaves and twigs. If the filter is clean, the pit may need to be reconstructed to maintain its performance.

7.4.8 Perforated Pipes

Perforated-pipe systems cannot be maintained as most conventional stormwater control facilities. If the perforated-pipe system fails the only recourse available is usually to reconstruct the pipe system. It is, therefore, very important to provide upstream sediment control facilities and to maintain those facilities as required to decrease the potential for plugging of the perforated pipe system. Other normal municipal maintenance activities such as street sweeping will also reduce the potential for clogging of perforated-pipe systems.

Catchbasin cleaning and regular maintenance of oil/grit separators may also increase the life span of perforated-pipe systems.

Although the effectiveness of perforated-pipe flushing has not been studied to any significant degree there are three methods that have been used by various municipalities. These include:

- Flushing: Most municipalities are familiar with sewer flushing. Sewer flushing is generally undertaken to clean out material that has been deposited in the pipe. It is anticipated that clogging will occur at the interface of the perforated pipe and the surrounding backfill/storage if the pipe is not wrapped in filter cloth and at the interface of the pipe and the filter material if the pipe is wrapped in filter cloth. If clogging occurs at the interface of the pipe and the filter material, sewer flushing may not prevent clogging in these systems.

- Radial Washing: Radial washing is similar in operation to flushing. The perforated pipe must be connected between manholes and the downstream end plugged or capped. A water hose is connected to the upstream end of the perforated pipe and water is introduced from the surface into the hose. The perforated pipe is essentially pressurized forcing water out through the perforations and hence, cleaning plugged perforations. Radial washing can be performed after flushing if there is considerable sediment deposition in the pipe itself.

- Jet Flushing: Jet flushing is frequently used in leachate collection systems for landfills to clean the perforated collection pipes. A pressurized hose is attached to an end nozzle that discharges water in various directions to clean the pipe. The pressure in the pipe on the end nozzle also directs the hose further along the pipe (self-directing). There are various nozzle designs available, and one that directs water radially into the perforations would be appropriate for perforated storm sewer applications.

7.4.8.1 Inspections

The following inspections should be conducted:

- Are the pretreatment SWMPs operating properly? Pretreatment SWMPs should be inspected (see oil/grit separators, grassed swales).
- Is the perforated pipe operating properly? The connection to the perforated pipe (that is, manhole/catchbasin) should be visually inspected for standing water 24 hours after a storm. Standing water will indicate the need for maintenance of the perforated-pipe system (flushing, jet washing).

7.4.9 Pervious Catchbasins

Pervious catchbasins normally discharge to an infiltration trench or to surrounding soils. The pervious catchbasin acts as a sediment trap and must be frequently cleaned out to protect the discharge area from clogging. Pervious catchbasins are the primary stormwater interceptor and as such are subject to high sediment loads and potentially clogging oils. The long-term infiltration potential is dependent on the quality of upstream flow. Clogged pervious catchbasins normally require reconstruction.

7.4.7.1 Inspections

The following inspections should be conducted for pervious catchbasins:

- Is sediment collecting in the catchbasin? The level of sediment in the catchbasin sump should be measured using a gradated pole with a flat plate attached to the bottom. The pole should be graduated such that the true bottom of the catchbasin compared to the grate is marked for comparison.

- Is there oil in the catchbasin? A visual inspection of the catchbasin contents should be made from the surface for trash/debris and/or the presence of an oil/industrial spill. An oily sheen, or frothing or unusual colouring to the water will indicate the occurrence of an oil or industrial spill. The separator/catchbasin should be cleaned in the event of spill contamination.

- Is the catchbasin infiltration system operating properly? The catchbasin should be visually inspected for standing water near the invert of the regular storm sewer 24 hours after a storm.

7.4.10 Grassed Swales

Grassed swales convey stormwater to other management facilities or to receiving streams. The swales however are designed to infiltrate a significant component of the stormwater flow prior to final discharge from the swale. Maintenance includes debris removal and sediment removal to ensure the infiltration effectiveness and capacity of the swale. The vegetative cover in the swale is also important to reduce flow velocities, reduce soil compaction (which in turn reduces rates of infiltration), and reduce erosion. Maintenance of grassed swales should include revegetation of any denuded areas.

7.4.10.1 Inspections

The following inspections should be conducted:

- Is there standing water in an enhanced grass swale? This will indicate a blocked check dam or decrease in the permeability of the swale. The check dam should be inspected for blockage by trash/debris or sediment.

- Is the grass/vegetation dead? This will indicate the need to revegetate the swale.

- Is there erosion downstream of the swale? This may indicate frequent overtopping of the swale, and as such, blockage of the dam or decreased swale permeability. The dam should be inspected for blockage and the erosion corrected by sodding. There may be a need to provide further erosion control (riprap, plant stakings) to prevent the reoccurrence of erosion.

7.5 Capital and Operating Costs of Stormwater BMP's

Capital costs and operating costs of stormwater BMP's are difficult to estimate from reported construction and maintenance activities in other locations. Most BMP's have very site-specific requirements that are a function of the stormwater quality, the local conditions and design objectives, as well as environmental considerations, land uses, and public preferences. Also, costs vary from one location to another as a function of the local economies. Capital and operating costs of any particular BMP can be expected to have a great deal of variability.

Despite this variability, the experience of other municipalities in constructing and maintaining stormwater BMP's can be useful to designers in their efforts to select appropriate stormwater BMP's and arrive at estimates for construction and maintenance at a planning or feasibility level. At a design stage, it is necessary to examine unit costs on a site-by-site basis for each selected BMP.

7.5.1 Cost Components

The total cost of implementing a stormwater BMP includes a number components including costs for administration, planning and design, land acquisition, site preparation, site development, and operation and maintenance.

Capital costs: The total cost including labour and materials associated with the actual onsite construction of the BMP facility.

Engineering costs: The total cost including labour and expenses for the planning and final design of the BMP. Engineering costs are normally estimated at 10 percent of the total capital cost.

Operation and maintenance costs: The total labour and expense cost associated with operating and maintaining the BMP at an acceptable level of performance.

Contingency costs: The cost associated with unforseen construction elements that are required over the construction period. Contingency costs are normally estimated at 15 percent of the total capital cost.

7.5.2 Source Control Costs

The costs associated with the implementation of source BMP's will vary from municipality to municipality and from site to site. The implementation costs experienced by various municipalities can, however, provide a basis for planning and evaluating stormwater BMP's.

Street sweeping can be a relatively labour-intensive and capital-intensive management practice. Street-sweeping costs are a function of the frequency of sweeping required for street surfaces, the equipment required, disposal costs, and operation and maintenance. The optimum sweeping frequency for any particular street is dependent on the surrounding land use but is usually, at a minimum, in the spring and fall. Sweeping frequency can be increased to as much as once per day for commercial core areas. Capital equipment costs vary because each municipality has specific requirements but are generally in the range of $65,000 to $120,000 (1991 US Dollars) (SEWRPC, 1991). Operation and maintenance costs are dependent on the frequency of street-sweeping. Unit costs associated with various street-sweeping programs are reported in Table 7-1.

Table 7-1
Reported Unit Costs for Street Sweeping Programs

Cost Factor	Nationwide Urban Runoff Program (NURP) Studies				San Jose, California (Pitt, 1979)	City of Milwaukee (1988)	Mean of all Studies
	Milwaukee, Wisconsin	Winston-Salem, Forsyth County, North Carolina	San Francisco Bay Area, California	Champaign, Illinois			
Cost per Pound of Solids Collected	NA	0.17-0.93	0.12-0.34	NA	0.05-0.32	NA	0.32
Cost per cubic yard of Solids Collected	NA	NA	NA	NA	40.0	13.4	26.7
Cost per Curb-Mile Swept	25.0	17.9	12.9-19.4	14.3-18.0	27.2	25.0	21.2
Cost per Hour of Sweeping Operation	36.0	21.8-46.6	NA	NA	29.7	NA	33.3

NA Indicates data not available

NOTE: All costs are 1989 $US

Source: SEWRPC. 1991

Catchbasin cleaning is carried out generally once per year or once every 2 years. The required frequency is a function of the surrounding land use and drainage characteristics within a given catchment area. The basins are cleaned manually or vacuumed using a vacuum truck or attachment for a vacuum street sweeper. The reported cost for manually cleaning a catchbasin is between $5 and $15 in 1984.

Anti-litter regulation costs are associated with the initial drafting of an anti-litter bylaw that can be used to enforce pet-owner responsibilities for their animals and actual enforcement of the bylaw. These costs vary considerably as a function of the size of the municipality and available resources.

7.5.3 Costs of Implementation

There is a lack of data for BMP capital costs. Information that is available is generally in the form of unit costs associated with different types of construction activities and materials. Typical unit costs applicable to Alberta are presented in Table 7-2. These general unit costs can be used as a guideline when preliminary costs are required for the evaluation of BMP's. To determine an approximate capital cost, estimate the quantities of each of the capital cost items and apply the unit cost (capital cost is the product of the unit price and the required quantity).

Table 7-2
Typical Unit Costs for Capital Construction (1996)

Type of Construction or Material	Unit	Price
Excavation (offsite disposal)	m^3	$10
Earthwork (cut and fill onsite)	m^3	$3
Erosion block/stone	m^2	$50
Concrete Outlet Structure	each	$5,500
Concrete Outlet Pipe (300 mm/600 mm/900 mm	m	$70/$170/$300
Observation Well (100 mm PVC)	each	$15
Riprap (450 mm)	m^2	$50
Perforated Pipe (100 mm, plastic)	m	$10
Perforated Riser Outlet Pipe (300 mm, plastic)	m	$90
Perforated Riser Outlet Trash Rack (400 CMP)	m	$100
Temporary Fencing (post and wire)	m	$15
Concrete (poured)	m^3	$600
Trash Rack (metal)	m^2	$100
Inverted Elbow Pipe	each	$300
Outlet Gate Valves (300 mm/600 mm)	each	$1,200/$4,800
Outlet Sluice Gates (300 mm/600 mm/900 mm)	each	$5,500/$8,000/$11,500
Clear Stone (gravel, 25 mm - 50 mm)	m^3	$35

Table 7-2 Typical Unit Costs for Capital Construction (1996)		
Type of Construction or Material	**Unit**	**Price**
Filter cloth	m^2	$1
Filter Material (sand)	m^3	$15
Structures	each	$50,000/facility
Monitoring Equipment	each	$20,000/facility
Sub-drainage System	ha	$25,000
Irrigation System	m^2	$2
Geomembrane Liner	m^2	$10 (dependent upon soil conditions
Seed and Topsoil	m^2	$2.50
Grass Sod and Topsoil	m^2	$4.50
Emergent and Submergent Fringe Vegetation	m^2	$12
Shoreline Fringe and Flood Fringe Vegetation	m^2	$12
Upland Vegetation	m^2	$5
Trees (Wooded Filter Strips)	m^2	$25

Information available in Alberta for total capital costs for BMP's is limited to wet/dry retention ponds. Typical costs in Calgary for a dry pond range from $5,000 to in excess of $25,000 per ha of catchment. For example a pond for a 100-ha catchment could range in cost from $0.5 million to $2.5 million. Wet ponds will be in the same price range except that additional costs for erosion protection within the pond would need to be allowed for. These costs do not include land costs, acreage assessment costs, irrigation, weeping tiles, inlet and outlet structures, etc. These costs can be even greater if there are special requirements, for example one dry pond in Calgary costs almost $50,000 per ha of development.

As with most construction activities, there are economies of scale that must be considered in using unit costs to arrive at preliminary costs estimates. There are also aesthetic considerations and safety considerations that can add to the basic cost of construction. The phasing of construction activities can also be an important factor in the overall cost. For preliminary planning purposes there are a number of general guidelines that are applicable to most municipalities and that can be used in the evaluation of BMP's based on cost:

- Facilities such as extended detention dry basins and wet ponds normally have much larger volume requirements than infiltration trenches and porous pavements.

- There are significant economies of scale for extended-detention dry basins, wet ponds, and surface infiltration trenches. However, infiltration trenches are probably not economically or physically feasible for large detention volumes. Economies of scale are less apparent for underground facilities.

- The cost of underground facilities is notably higher than the costs of other BMP's.

- Wet ponds are more expensive to construct than extended-detention dry basins probably due to the greater excavation requirements of the permanent pool in wet ponds. However, wet ponds appear to have greater economies of scale than dry basins.

- Since excavation at the construction phase is generally less expensive than dredging of accumulated sediments at a later date, it is economical to design BMP's with excess sediment storage capacity. It is also economical to reserve sediment disposal areas onsite.

- Infiltration facilities that are designed to incorporate extra runoff storage volumes are relatively expensive. It is therefore more economical to bypass excess flows from infiltration facilities to other BMP's designed for detention.

- Based solely on costs, extended-detention dry basins are the most economical BMP for most applications. However, they provide poor water quality controls. Infiltration basins are also relatively cost-effective especially in areas of intensive development.

- The cost-effectiveness of treatment BMP's depends on the perceived value of other aspects of multi-use facilities as well as design and construction costs and pollutant removal effectiveness.

7.5.4 Costs of Operation and Maintenance

Appropriate operation and maintenance budgets are an essential component of all stormwater BMP's. The unit costs of operation and maintenance are also difficult to directly determine based on the experience of other municipalities. As with capital costs, operation and maintenance costs can be expected to be quite variable from municipality to municipality and from site to site because of differences in drainage basin characteristics of runoff and sediment load, labour and equipment costs, disposal costs, and design/performance objectives for the facility.

The costs presented in Table 7-3 represent typical unit costs of a number of operation and maintenance activities associated with various BMP's including vegetation management and sediment control. These general unit costs can be used as a guideline when preliminary costs are required for the evaluation of BMP's.

A total annual budget of 3 to 5 percent of the total construction costs should be allowed for operation and maintenance of most BMP's. Infiltration trenches are an exception to this with an allowance recommended of 5 to 10 percent of construction costs for surface facilities and 10 to 15 percent for underground facilities. Operating costs should also include provisions for ongoing performance monitoring of the BMP in order to optimize operation and maintenance requirements as well as to determine the effectiveness of the BMP in enhancing hydrologic and water quality conditions.

Table 7.3
Typical Unit Costs for Operations and Maintenance

Type of Maintenance	Unit	Ontario		City of Edmonton	
		Price	Maintenance Interval	Price	Maintenance Interval
Litter Removal	ha	$2,000	Every year	-	-
Grass Cutting	ha	$250	***	-	-
Weed Control	ha	$2,500	Every year	$2,400	1 to 3 times per year
Vegetation Maintenance (Aquatic/Shoreline Fringe)	ha	$3,500	Every 5 years	-	-
Vegetation Maintenance (Upland/Flood Fringe)	ha	$1,000	Every 5 years	-	-
Sediment Removal (front end loader)	m^3	$15	*	-	-
Sediment Removal (vacuum truck - catch basin, filter strip, grassed swale)	m^3	$120	*	-	-
Sediment Removal (manual - oil/grit separator, sand filter)	m^3	$120	*	-	-
Sediment testing (lab tests for quality)	each	$265	*	-	-
Sediment Disposal (off-site landfill)	m^3	$300	*	-	-
Sediment Disposal and Landscaping (on-site)	m^3	$5	*	-	-
Inspection (inlet/outlet etc.)	each	$100	Every year	-	Early spring and every visit to pond
Pervious Pipe Cleanout (flushing)	m	$1	Every 5 years	-	-
Pervious Pipe Cleaning (Radial Washing)	m	$2	Every 5 years	-	-
Seasonal Operation of Infiltration System By-pass	**	$100	Twice per year	-	-
Infiltration Basin Floor Tilling and Re-vegetation	ha	$2,800	Every 2 years	-	-
Water Sampling	each	-	-	-	Monthly (Spring to Fall only)
Remove Shoreline Debris	each	-	-	-	Twice per month
Remove Floating Debris	each	-	-	-	As required
Check Depth of Sediment	each	-	-	-	Every 2 to 5 years
Routine Maintenance	each	-	-	$3,800	-
Vegetation Harvesting	ha	-	-	-	Once per year (Late Fall)

* frequency of sediment removal depends on SWMP type and volume
** dependent on filtration facility (based on centralized facility) Seasonal operation of a system with many inlets (ie pervious pipe system) would be more expensive
*** no grass cutting or minimal frequency of grass cutting (once or twice per year)

Source: Ontario Ministry of Environment and Energy; SWMP Planning and Design Manual, 1994
Personal correspondence from City of Edmonton, 1996